**Page**

# 5.0 Reinforced Concrete Superstructures

## 5.1 General

Prior to precast pretensioned and post-tensioned concrete members introduced in the early 1960s, all short and medium span bridges were built as cast-in-place (CIP) reinforced concrete superstructures.

Examples of reinforced concrete superstructures are: flat slabs, slab and T-beams, arches, slabs for all types of steel bridges, and box girders.

Many of the bridges built before 1960 are functional, durable, and structurally sound. The service life of some of these early bridges can be extended by widening their decks to accommodate increased traffic demand or to improve safety. This chapter addresses special requirements for widenings.

The design aids in this chapter can also be utilized in the design of nonprestressed reinforcement in prestressed structural elements and reinforced concrete substructures.

### 5.1.1 Concrete and Grout

A. Classes of Concrete

1. CLASS 3000

   Used in large sections with light to nominal reinforcement, mass pours, sidewalks, curbs, gutters, and nonstructural concrete guardrail anchors, luminaire bases.

2. CLASS 4000

   Used in cast-in-place post-tensioned or reinforced concrete box girders, slabs, traffic and pedestrian barriers, approach slabs, footings, box culverts, wing walls, curtain walls, retaining walls, columns, and crossbeams.

3. CLASS 4000D

   Used in bridge concrete decks. Standard specifications require two coats of curing compound and a continuous wet cure for 14 days.

4. CLASS 4000P

   Used for cast-in-place pile and shaft.

5. CLASS 4000W

   Used underwater in seals.

6. CLASS 5000 or Higher

   Used in CIP post-tensioned concrete box girder construction or in other special structural applications situations. Use of CLASS 5000 or higher requires approval of the Bridge Design Engineer, the Olympia Service Center, and Materials Lab. Place documentation in job file.

B. Strength of Concrete

1. The 28-day compressive design strengths ($f_c'$) in pounds per square inch (psi) are:

| Class | $f_c'$ |
|---|---|
| COMMERCIAL | 2300 |
| 3000 | 3000 |
| 4000, 4000D | 4000 |
| 4000W | 2400* |
| 5000 | 5000** |
| 6000 | 6000 |
| 4000P | 3400***-shaft |
| 4000P | 4000-piles |

*40 percent reduction from CLASS 4000.

**Concrete Class 5000 is available within a 30-mile radius of Seattle, Spokane, and Vancouver. Outside this 30-mile radius, concrete suppliers do not have the quality control rocedures and expertise to Supply Control Class 5000.

***15 percent reduction from 4000 psi for all drilled shafts. All drilled shafts are assumed to be constructed in wet conditions, that warrants reduction in concrete design strength.

2. Relative Compressive Concrete Strength

   a. During design or construction of a bridge, it is necessary to determine the strength of concrete at various stages of construction. For instance, Section 6-02.3(17)N of the *Standard Specifications* discusses the time at which falsework and forms can be removed to various percentages of the concrete design strength. Occasionally, construction problems will arise which require a knowledge of the relative strengths of concrete at various ages. Table 5.1-1 is intended to supply this information.

   b. Curing conditions of the concrete (especially in the first 24 hours) have a very important influence on the strength development of concrete at all ages. Temperature affects the rate at which the chemical reaction between cement and water takes place. Loss of moisture can seriously impair the concrete strength.

   c. Table 5.1-1 shows the approximate values of the minimum compressive strengths of different classes of concrete at various ages. If the concrete has been cured under continuous moist curing at an average temperature, it can be assumed that these values have been developed.

   d. If test strength is above or below that shown in Table 5.1-1, the age at which the design strength will be reached can be determined by direct proportion.

   For example, if the relative strength at 10 days is 64 percent instead of the minimum 70 percent shown in Table 5.1-1, the time it takes to reach the design strength can be determined as follows:

   Let x = relative strength to determine the age at which the concrete will reach the design strength

   $$\frac{x}{70} = \frac{100}{64}$$ Therefore, x = 110

   From Table 5.1.1-1, the design strength should be reached in 40 days.

C. Grout

Grout is usually a prepackaged cement based grout or nonshrink grout that is mixed, placed, and cured as recommended by the manufacturer. It is used under steel base plates for both bridge bearings and luminaire or sign bridge bases. Nonshrink grout is used in keyways between precast prestressed deck slabs, tri-beams, and bulb-tees. For design purposes, the strength of the grout, if properly cured, can be assumed to be equal to or greater than that of the adjacent concrete.

Should the grout pad thickness exceed 4 inches, steel reinforcement shall be used.

The following chart shows approximate relative strength of concrete and compressive strength of different classes of concrete at various ages based on continuous moist curing at an average temperature.

Relative and Compressive Strength of Concrete
Table 5.1.1-1

| Age (Days) | Relative Strength (%) | Class 5000 (psi) | Class 4000 (psi) | Class 3000 (psi) | Age (Days) | Relative Strength (%) | Class 5000 (psi) | Class 4000 (psi) | Class 3000 (psi) |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 35 | 1750 | 1400 | 1050 | 20 | 91 | 4550 | 3640 | 2730 |
| 4 | 43 | 2150 | 1720 | 1290 | 21 | 93 | 4650 | 3720 | 2790 |
| 5 | 50 | 2500 | 2000 | 1500 | 22 | 94 | 4700 | 3760 | 2820 |
| 6 | 55 | 2750 | 2200 | 1650 | 23 | 95 | 4750 | 3800 | 2850 |
| 7 | 59 | 2950 | 2360 | 1770 | 24 | 96 | 4800 | 3840 | 2880 |
| 8 | 63 | 3150 | 2520 | 1890 | 25 | 97 | 4850 | 3880 | 2910 |
| 9 | 67 | 3350 | 2680 | 2010 | 26 | 98 | 4900 | 3920 | 2940 |
| 10 | 70 | 3500 | 2800 | 2100 | 27 | 99 | 4950 | 3960 | 2970 |
| 11 | 73 | 3650 | 2920 | 2190 | 28 | 100 | 5000 | 4000 | 3000 |
| 12 | 75 | 3750 | 3000 | 2250 | 30 | 102 | 5100 | 4080 | 3060 |
| 13 | 77 | 3850 | 3080 | 2310 | 40 | 110 | 5500 | 4400 | 3300 |
| 14 | 79 | 3950 | 3160 | 2370 | 50 | 115 | 5750 | 4600 | 3450 |
| 15 | 81 | 4050 | 3240 | 2430 | 60 | 120 | 6000 | 4800 | 3600 |
| 16 | 83 | 4150 | 3320 | 2490 | 70 | 125 | 6250 | 5000 | 3750 |
| 17 | 85 | 4250 | 3400 | 2550 | 80 | 129 | 6450 | 5160 | 3870 |
| 18 | 87 | 4350 | 3480 | 2610 | 90 | 131 | 6550 | 5240 | 3930 |
| 19 | 89 | 4450 | 3560 | 2670 | | | | | |

### 5.1.2 Reinforcement

A. Grades

Steel reinforcing bars are manufactured as plain or deformed bars (which have ribbed projections that grip the concrete in order to provide better bond between steel and concrete). In Washington State, main bars are always deformed. Plain bars maybe used for spirals and ties.

Reinforcing bars conform to either the requirements of AASHTO M31, Grade 60 (ASTM A-615 Grade 60) with a 60,000 psi yield strength or ASTM A 706 Specifications for Low-Alloy Steel deformed Bars for Concrete Reinforcement in the case of bars in portions of concrete members where plastic hanging can occur during an earthquake or which are to be spliced by welding,

B. Sizes

Reinforcing bars are referred to in the contract plans and specifications by number and vary in size from #3 to #18. For bars up to and including #8, the number of the bar coincides with the bar diameter in eighths of an inch. The #9, #10, and #11 bars have diameters that provide areas equal to 1″ x 1″ square bars, $1^1/_8$″ x $1^1/_8$″ square bars and $1^1/_4$″ x $1^1/_4$″ square bars respectively. Similarly, the #14 and #18 bars correspond to $1^1/_2$″ x $1^1/_2$″ and 2″ x 2″ square bars, respectively. Tables 5.1-A1 through 5.1-A3 in Appendix A, show the sizes, number, and various properties of the types of bars used in Washington State.

C. Development

1. Development Length, $l_d$, in Tension

Development length or anchorage of reinforcement is required on both sides of a point of maximum stress at any section of a reinforced concrete member.

Development of bars in tension involves calculating the basic development length, $l_{db}$, which is modified by factors to reflect bar spacing, cover, enclosing transverse reinforcement, top bar effect, type of aggregate, epoxy coating, and ratio of required area to provided area of reinforcement to be developed.

The development length, $l_d$ (including all applicable modification factors) must not be less than 12 inches.

Tables 5.1-A4 and 5.1-A5 in Appendix A, show the tension development length for both uncoated and epoxy coated Grade 60 bars for normal weight concrete with specified strengths of 3,000 to 6,000 psi.

2. Development Length, $l_d$, in Compression

The basic development lengths for deformed bars in compression are shown in Table 5.1-A7, Appendix A. These values may be modified for ratio of required area vs. provided area of reinforcement, or for bars enclosed in a $^1/_4$ inch diameter spiral at 4 inch maximum pitch. However, the minimum development length is 1 foot 0 inches (office practice).

3. Standard End Hook Development Length, $l_{dh}$, in Tension

Standard end hooks, utilizing 90 and 180 degree end hooks, are used to develop bars in tension where space limitations restrict the use of straight bars. End hooks on compression bars are not effective for development length purposes. Figures 5.1.2-1 and 5.1.2-2 and Table 5.1.2-1 show the minimum embedment lengths necessary to provide 2 inches of cover on the tails of 90 and 180 degree end hooks. Epoxy coating does not affect the tension development lengths, $l_{dh}$, of standard 90 and 180 degree end hooks. The values shown in Table 5.1-1A5, Appendix A, show the tension development lengths for normal weight concrete with specified strengths of 3,000 to 6,000 psi.

D. Splices

Three methods are used to splice reinforcing bars; lap splices, mechanical splices, and welded splices. Lap splicing of reinforcing bars is the most common method. The Contract Plans should clearly show the locations and lengths of lap splice. Lap splices are not permitted for bars larger than #11.

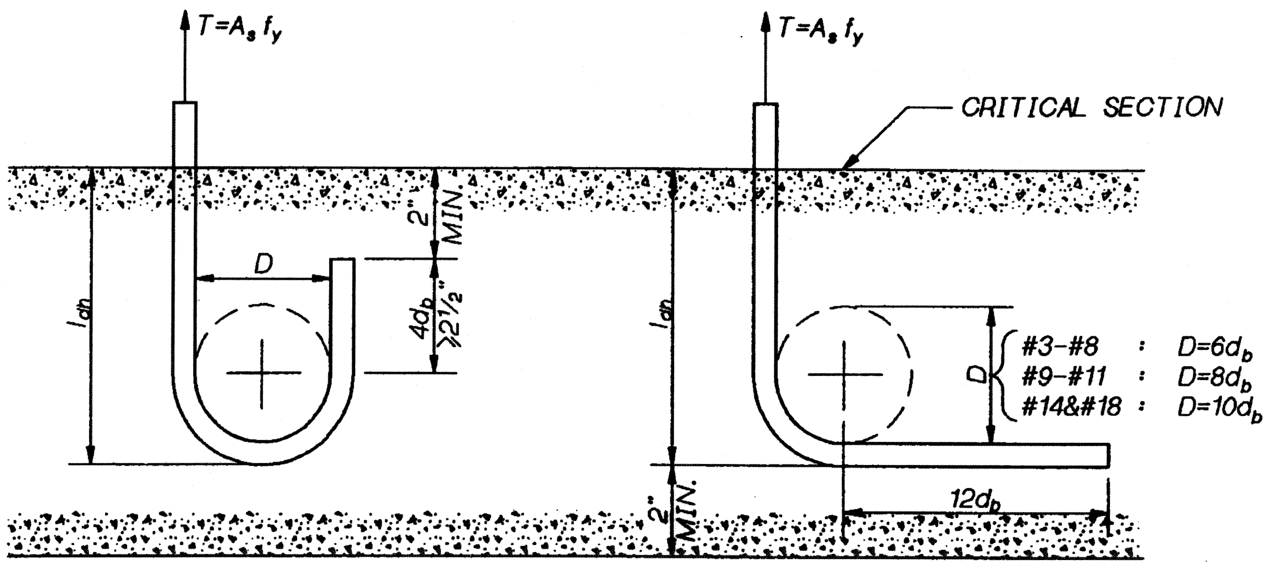
No lap splices, for either tension or compression bars, shall be less than 2 feet 0 inches (office practice). See Section 8.32 of the *Standard Specifications for Highway Bridges* and Section 6-02.3(24)D *Standard Specifications* for additional splice requirements.

1. Lap Splices — Tension

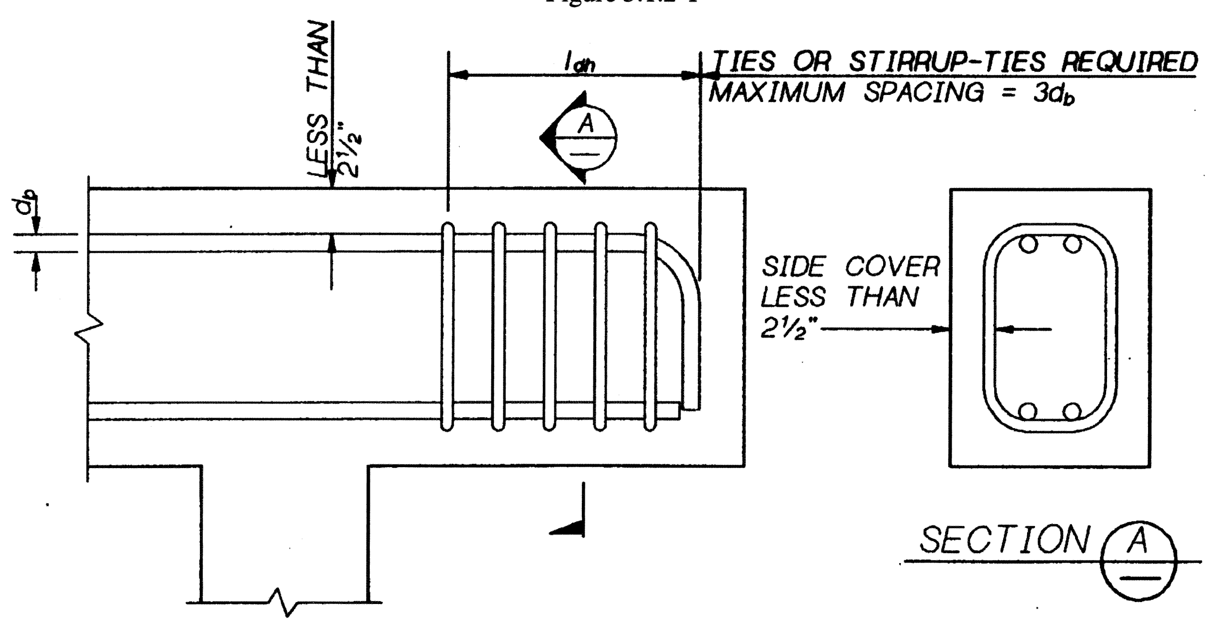
Many of the same factors which affect development length affect splices. Consequently, tension lap splices are a function of the bar's development length, $l_d$. There are three classes of tension lap splices: Class A, B, and C. Designers are encouraged to splice bars at points of minimum stress and to stagger lap splices along the length of the bars.

Minimum Embedment Lengths to Provide 2-inch Cover to Tail of Standard 180° End Hooks
Table 5.1.2-1

| #3 | #4 | #5 | #6 | #7 | #8 | #9 | #10 | #11 | #14 | #18 |
|---|---|---|---|---|---|---|---|---|---|---|
| 6″ | 7″ | 9″ | 10″ | 1′-0″ | 1′-2″ | 1′-3″ | 1′-5″ | 1′-7″ | 2′-10″ | 3′-7″ |




Standard 180° and 90° End Hooks
Figure 5.1.2-1



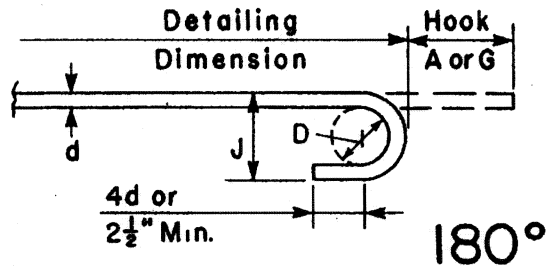

Special Confinement for 180° and 90° End Hooks
Figure 5.1.2-2

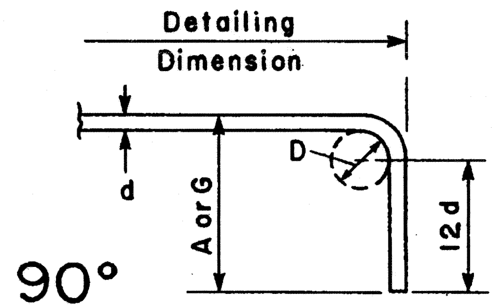
Standard Hooks
Table 5.1.2-2

All specific sizes recommended by CRSI below meet minium requirements of ACI 318.

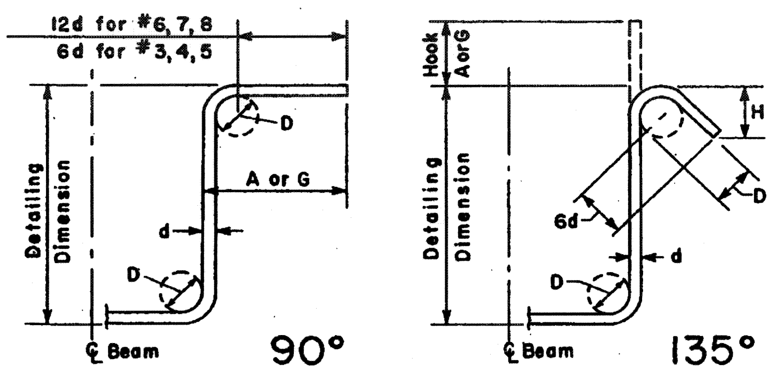
RECOMMEND END HOOKS
All Grades
D=Finished bend diameter

| Bar Size | D | 180° HOOKS | | 90° HOOKS |
|---|---|---|---|---|
| | | A or G | J | A or G |
| # 03 | 2 1/4 | 5 | 3 | 6 |
| # 04 | 3 | 6 | 4 | 8 |
| # 05 | 3 3/4 | 7 | 5 | 10 |
| # 06 | 4 1/2 | 8 | 6 | 1-0 |
| # 07 | 5 1/4 | 10 | 7 | 1-2 |
| # 08 | 6 | 11 | 8 | 1-4 |
| # 09 | 9 1/2 | 1-3 | 11 3/4 | 1-7 |
| # 10 | 10 3/4 | 1-5 | 1-1 1/4 | 1-10 |
| # 11 | 12 | 1-7 | 1-2 3/4 | 2-0 |
| # 14 | 18 1/4 | 2-3 | 1-9 3/4 | 2-7 |
| # 18 | 24 | 3-0 | 2-4 1/2 | 3-5 |



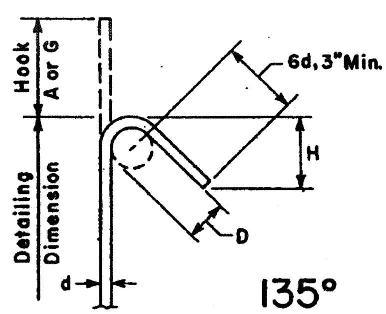

STIRRUP AND TIE HOOKS




STIRRUP
(TIE SIMILAR)
STIRRUP AND TIE HOOK DIMENSIONS
ALL GRADES

| Bar Size | D (in.) | 90° HOOKS | 135° HOOKS | |
|---|---|---|---|---|
| | | Hook A or G | Hook A or G | H Approx. |
| # 03 | 1 1/2 | 4 | 4 | 2 1/2 |
| # 04 | 2 | 4 1/2 | 4 1/2 | 3 |
| # 05 | 2 1/2 | 6 | 5 1/2 | 3 3/4 |
| # 06 | 4 1/2 | 1-0 | 8 | 4 1/2 |
| # 07 | 5 1/4 | 1-2 | 9 | 5 1/4 |
| # 08 | 6 | 1-4 | 10 1/2 | 6 |

135 ° SEISMIC STIRRUP/TIE HOOKS

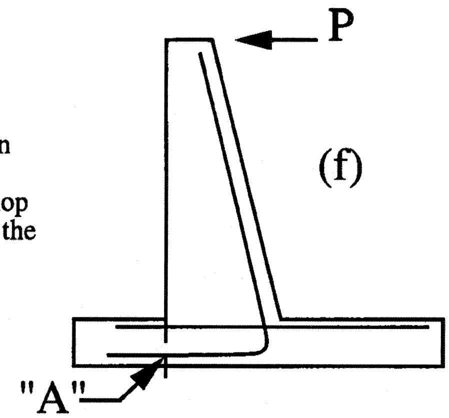



135 ° SEISMIS STIRRUP/TIE
HOOK DIMENSIONS
ALL GRADES

| Bar Size | D (in.) A or G | 135° HOOKS | |
|---|---|---|---|
| | | Hook A or G | Hook |
| # 03 | 1 1/2 | 4 1/4 | 3 |
| # 04 | 2 | 4 1/2 | 3 |
| # 05 | 2 1/2 | 5 1/2 | 3 3/4 |
| # 06 | 4 1/2 | 8 | 4 1/2 |
| # 07 | 5 1/4 | 9 | 5 1/4 |
| # 08 | 6 | 10 1/2 | 6 |

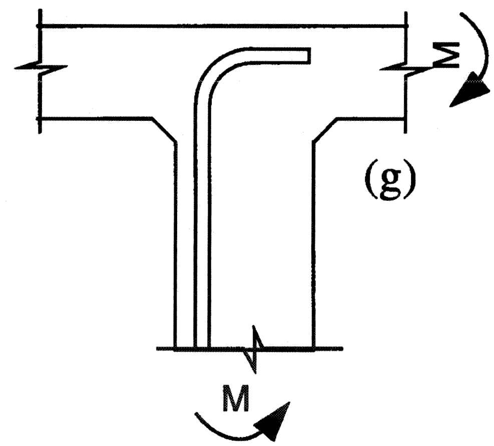
**Cantilever Retaining Wall**

The footing near the corner, at the junction between the stem and the footing, with reinforcing as in figure (f) will fully develop the resisting moment as long as the toe of the footing is long enough for anchorage, and stress at "A" (bottom) is not critical.




**T-Joint**

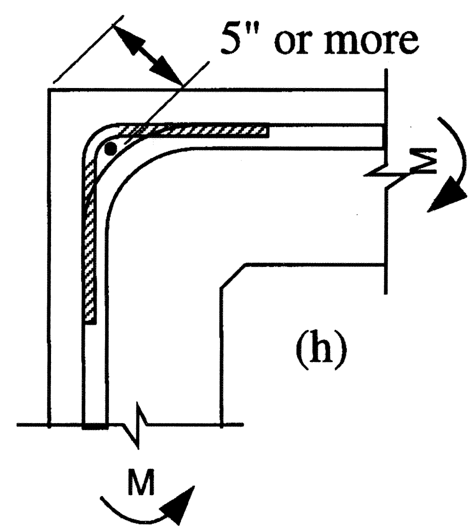
The forces in figure (g) form a tension crack at 45°. Reinforcement as shown is more than twice as effective in developing the strength of the corner than if the reinforcement were turned 180°.




**"Normal" right corners**

Corners subjected to bending as in figure (h) will crack radially in the corner outside of the main reinforcing steel. Smaller size reinforcing steel (similar to temperature reinforcing steel) shall be provided in the corner as shown to distribute the radial cracking.

For highly stressed reinforcing steel, the radius of bend may have to be increased to keep radial pressures against the concrete within safe limits.

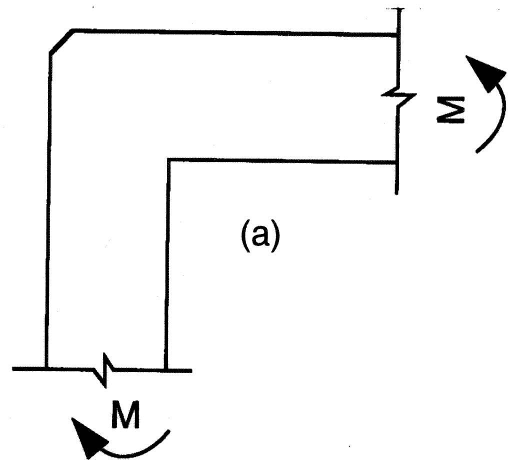



For a more detailed discussion, refer to the June 1976, ASCE Journal of the Structural Division, "Reinforced Concrete Corners and Joints Subjected to Bending Moment" by Inguar H.E. Nilsson and Anders Losberg. This paper is filed under 5.2 Reinforced Concrete Superstructures.

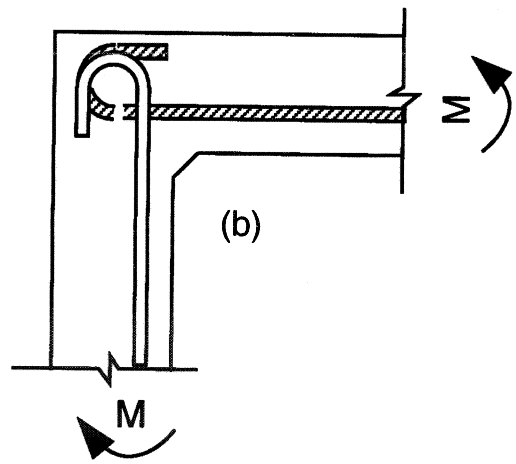
Figure 5.1.2-3

**Right or obtuse angle corners**

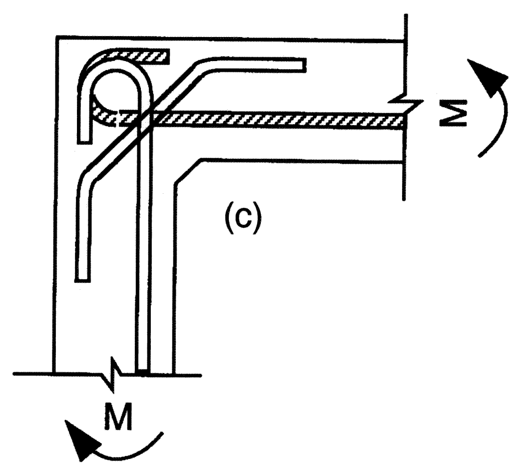
Corners subjected to bending as in figure (a) tend to crack at the reentrant corner and fail in tension across the corner. If not properly reinforced, the resisting corner moment is less than the applied moment.




Reinforced as is figure (b), the section will develop 85% of the ultimate moment capacity of the wall. If the bends were rotated 180°, only 30% of the wall capacity would be developed.




Adding diagonal reinforcing steel across the corner as in figure (c), approximately equal to 50% of the main reinforcing steel, will develop the corner strength to fully resist the applied moment. Extend the diagonal reinforcement past the corner each direction for anchorage (4 feet long for bars through $7 are adequate).




Since this bar arrangement will fully develop the resisting moment, a fillet in the corner is normally unnecessary.

Figure 5.1.2-4

Table 5.1A6 in Appendix A, shows tension lap splices for both uncoated and epoxy coated Grade 60 bars for normal weight concrete with specified strengths of 3,000 to 6,000 psi. For additional requirements, see Section 8.32.3 of the AASHTO *Standard Specifications for Highway Bridges*.

For Seismic Performance Categories C and D, Section 8.4.1(F) of the AASHTO *Standard Specifications for Seismic Design of Highway Bridges*, the lap splices for longitudinal column bars are permitted only within the center half of the column height and shall not be less than the lap splices given in Table 5.1-A6 in Appendix A, or 60 bar diameters whichever is greater.

Note that the maximum spacing of the transverse reinforcement (i.e., column ties) over the length of the splice shall not exceed the smaller of 4 inches or $^1/_4$ of the minimum column plan dimension.

2. Lap Splices — Compression

   The compression lap splices shown in Table 5.1-A7 (right-hand column) in Appendix A, are for concrete strengths greater than 3,000 psi. If the concrete strength is less than 3,000 psi, the compression lap splices should be increased by one third. Note that when two bars of different diameters are lap spliced, the length of the lap splice shall be the larger of the lap splice for the smaller bar or the development length of the larger bar.

3. Mechanical Splices

   A second method of splicing is by mechanical splices, which are proprietary splicing mechanisms. The requirements for mechanical splices are found in Section 6-02.3(24)F of the *Standard Specifications*, Sections 8.32.2 and 8.32.3 of the AASHTO *Standard Specifications for Highway Bridges*, and Section 8.4.1(F) of the *Standard Specifications for Seismic Design of Highway Bridges*.

4. Welded Splices

   Welding of reinforcing bars is the third acceptable method of splicing reinforcing bars. Section 6-02.3(24)E of the *Standard Specifications* describes the requirements for welding reinforcing steel. On modifications to existing structures, welding of reinforcing bars may not be possible because of the non-weldability of some steels. See Sections 8.32.2 and 8.32.3 of the AASHTO *Standard Specifications for Highway Bridges* and Section 8.4.1(F) of the *Standard Specifications for Seismic Design of Highway Bridges* for additional welded splice requirements.

E. Bends

For standard hooks and bend radii, see Table 5.1.2-2. Note that the tail lengths are greater for the 135° seismic tie hook than for the regular or nonseismic 135° tie hook. For field bending requirements, see Section 6-02.3(24)A of the *Standard Specifications*.

F. Fabrication Lengths

Reinforcing bars are normally stocked in lengths of 60 feet. They can also be fabricated in longer lengths.

The maximum overall bar lengths to be specified on the plans are:

| Bar Size | Maximum Length |
|---|---|
| #3 | 30′-0″ |
| #4, #5 | 40′-0″ |
| #6, through # 18 | 60′-0″ |

Where possible, specify lengths 60 feet and less for bar sizes #8 through #18. Because of placement considerations, the overall lengths of bar size #3 has been limited to 30 feet and bar sizes #4 and #5 to 40 feet. To use longer lengths, the designer should make sure that the bars can be placed and transported by truck. See Table 5.1-A1 in Appendix A.

G. Placement

Placement of reinforcing bars can be a problem during construction. Reinforcing bars are more than just lines on the drawing, they have size, weight, and volume. In confined areas, the designer should ensure that reinforcing bars can be placed. Sometimes it may be necessary to make a large scale drawing of reinforcement to look for interference and placement problems. If interference is expected, additional details may be required in the contract plans showing how to handle the interference and placement problems.

H. Percentage Requirements

There are several AASHTO requirements to ensure that minimum reinforcement is provided in reinforced concrete members.

1. Flexure

The reinforcement provided at any section should be adequate to develop a moment at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal weight concrete. The modulus of rupture for normal weight concrete is $7.5\sqrt{f_c'}$. This requirement may be waived if the area of reinforcement provided is at least one-third greater than that required by analysis. For additional minimum reinforcement required, see Section 8.17, AASHTO *Standard Specifications for Highway Bridges*.

2. Compression

For columns, the area of longitudinal reinforcement shall not exceed 0.08 nor be less than 0.01 of the gross area, Ag, of the section. Preferably, the ratio of longitudinal reinforcement should not exceed 0.04 of the gross area, Ag, to ensure constructibility and placement of concrete. If a ratio greater than 0.04 is used, the designer should verify that concrete can be placed. If for architectural purposes the cross section is larger than that required by the loading, a reduced effective area may be used. The reduced effective area shall not be less than that which would require 1percent of the longitudinal area to carry the loading. Additional lateral reinforcement requirements are given in Section 8.18, AASHTO *Standard Specifications for Highway Bridges*, and for plastic hinge zones, see Section 8.4.1(D), AASHTO *Standard Specifications for the Seismic Design of Highway Bridges*.

3. Other Minimum Reinforcement Requirements

For minimum shear reinforcement requirements, see Section 8.19 and for minimum temperature and shrinkage reinforcement, see Section 8.20, AASHTO *Standard Specifications for Highway Bridges*.

## 5.2 Design Methods

### 5.2.1 Strength Design Method

A. Design Philosophy

In the strength design method or ultimate strength method, the service loads are increased by load factors to obtain the ultimate design load. The structural members are then proportioned to provide the design ultimate strength. Several textbooks listed in the bibliography, which are excellent sources [1,2,3].

B. Flexure

The basic strength design requirement can be expressed as follows:

$$\text{Design Strength} \geq \text{Required Strength or } \phi\ M_n \geq M_u \tag{1}$$

For design purposes, the area of reinforcement for a singly reinforced beam or slab can be determined by letting:

$$M_u = \phi\ M_n = \phi\ [A_s\ (f_y)\ (d - a/2)] \tag{2}$$

However, if a $A_s(f_y)/(0.85)(f_c')(b)$ and $\rho = A_s/(b)(d)$ (3)

Equation (2) can be expressed as:

$$M_u/\phi\ (b)\ (d)^2 = \rho\ (f_y)\ [1 - 0.59\ (\rho)\ f_y/f_c'] \tag{4}$$

Tables 5.2-1 through 5.2-3 in Appendix 5.2-A1, -A2, and -A3, were prepared based on Eq (4) to quickly determine the amount of reinforcing steel required, $A_{s\ required}$, when $M_u$, $f_c'$, $f_y$, b, and d are known.

An alternate approach is to solve directly for $A_{s\ required}$ from:

$$A_{s\ required} = \frac{0.85\ f_c'\ (b)}{f_y}\left(d - \sqrt{d^2 - \frac{31.3725\ M_u}{f_c'\ (b)}}\right) \quad \text{where} \quad Mu = \text{kips} - \underline{\text{ft}},\ f_c' = \text{ksi} \tag{5}$$

Similarly, substituting 1.2Mcr for $M_u$, $A_{s\ min}$ can be found from:

$$A_{s\ min} = \frac{0.85\ f_c'\ (b)}{f_y}\left(d - \sqrt{d^2 - \frac{0.124\ h^2}{\sqrt{f_c'}}}\right) \quad \text{where} \quad h = \text{slab thickness} \tag{6}$$

From AASHTO 8.16.3.1.1 and 8.16.3.2.2, $A_{s\ max}$ can be found from:

$$A_{s\ max} = 0.6375\ \beta_1\ (b)\ (d)\ \frac{f_c'}{f_y}\left(\frac{87}{87 + f_y}\right) \tag{7}$$

where $\beta_1 = 0.85$ if $f_c' \leq 4$ ksi and
$\beta_1 = 0.85 - 0.05\ (f_c' - 4)$ if $f_c' > 4$ ksi, but not less than 0.65

Tension reinforcement should be designed in the following order:

1. From Eq (5) or Tables 5.2-A1 through 5.2-A3 in Appendix A, determine $A_{s\ required}$.
2. From Eq (6) determine $A_{s\ min}$.
3. From Eq (7) or Tables 5.2-A1 through 5.2-A3 in Appendix A, determine $A_{s\ max}$.

4. If $A_{s\ required} > A_{s\ max}$, increase the member's dimensions.
   If $A_{s\ max} > A_{s\ required} > A_{s\ min}$, use $A_s \geq A_{s\ required}$.
   If $A_{s\ required} < A_{s\ min} < 1.33\ A_{s\ required}$, use $As \geq A_{s\ min}$.
   If $1.33\ A_{s\ required} < A_{s\ min}$, use $As \geq 1.33\ A_{s\ required}$.
   Always use $As \leq A_{s\ max}$.

See Appendix 5.2-B1 and 5.2-B2 for design examples.

C. Shear

The AASHTO *Standard Specifications for Highway Bridges* addresses shear design of members in Section 8.16.6. Shear friction provisions (Section 8.16.6.4) are applied to transfer shear across a plane, such as: an existing or potential crack, an interface between dissimilar materials, or at a construction joint between two sections of concrete placed at different times.

The shear design for deep beams is not addressed in the AASHTO Standard Specifications, but is discussed in Section 11.8, ACI 318-89 *Building Code Requirements for Reinforced Concrete and Commentary*, and ACI-ASCE Committee 343 *Analysis and Design of Reinforced Concrete Bridge Structures* [4,5,6].

D. Strut-and-Tie Model

1. General

Strut-and-tie models may be used to determine internal force effects near supports and the points of application of concentrated loads [16].

The strut-and-tie model should be considered for the design of deep beams and pile caps or other situations in which the distance between the centers of applied load and supporting reaction is less than twice the member thickness.

2. Structural Modeling

The structure and a component or region, thereof, may be modeled as an assembly of steel tension ties and concrete compressive struts interconnected at nodes to form a truss capable of carrying all the applied loads to the supports as shown in Figure 5.2.1-1 for a deep beam. The required widths of compression struts and tension ties shall be considered in determining the geometry of the truss. The truss model does not necessarily need to conform to structural stability as a real truss would.

The factored resistance, $P_n$,of struts and ties shall be taken as that of axially loaded components.

$$P_u' = \varphi P_n$$

where:

$P_n$ = nominal resistance of strut or tie (kip)
$\varphi$ = 0.7 Compression
$\varphi$ = 0.9 Tension

3. Proportioning of Compressive Struts

a. Strength of Unreinforced Strut

The nominal resistance of an unreinforced compressive strut shall be taken as:

$$P_n = f_{cu} A_{cs}$$

where:

$P_n$ = nominal resistance of a compressive strut (kips)

$f_{cu}$ = limiting compressive stress (ksi)

$A_{cs}$ = effective cross-sectional area of strut ($in^2$)

b. Effective Cross-Sectional Area of Strut

The value of $A_{cs}$ shall be determined by considering both the available concrete area and the anchorage conditions at the ends of the strut, as shown in Figure 5.2.1-2.

When a strut is anchored by reinforcement, the effective concrete area may be considered to extend a distance of up to six bar diameters from the anchored bar, as shown in Figures 5.2.1-2(a), 5.2.1-2(b), and 5.2.1-2(c).

c. Limiting Compressive Stress in Strut

The limiting compressive stress, $f_{cu}$, shall be taken as:

$$f_{cu} = \frac{f_c'}{0.8 + 170\varepsilon_1} \leq 0.8 f_c'$$

for which:

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002) \cot^2 \alpha_s$$

where:

$a_s$ = the smallest angle between the compressive strut and adjoining tension ties (DEG)
$\varepsilon_s$ = the tensile strain in the concrete in the direction of the tension tie (in/in)
$f_c'$ = specified compressive strength (ksi)

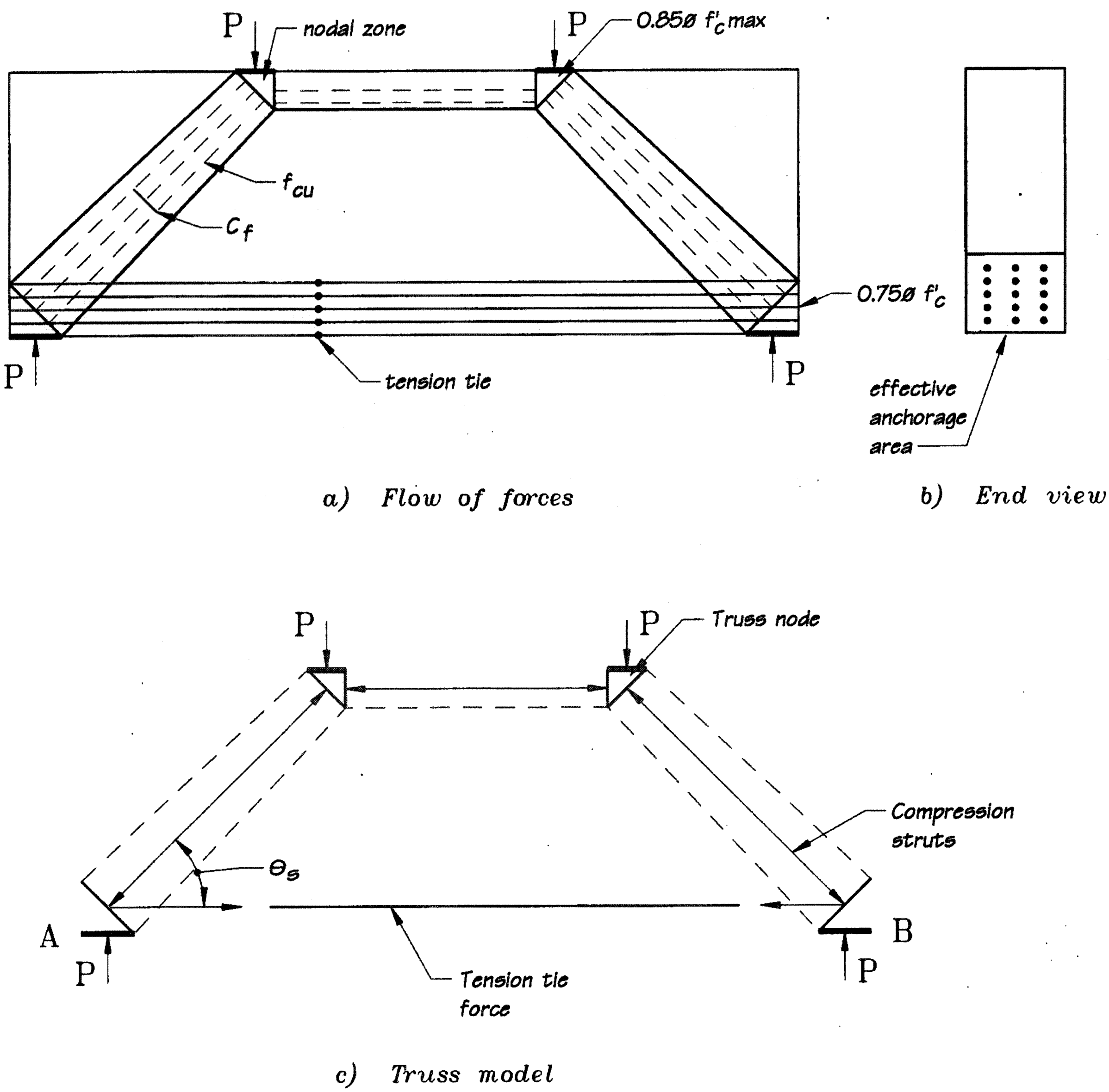
d. Reinforced Strut

If the compressive strut contains reinforcement that is parallel to the strut and detailed to develop its yield stress in compression as shown in Figure 5.2.1-2(d), the nominal resistance of the strut shall be taken as:

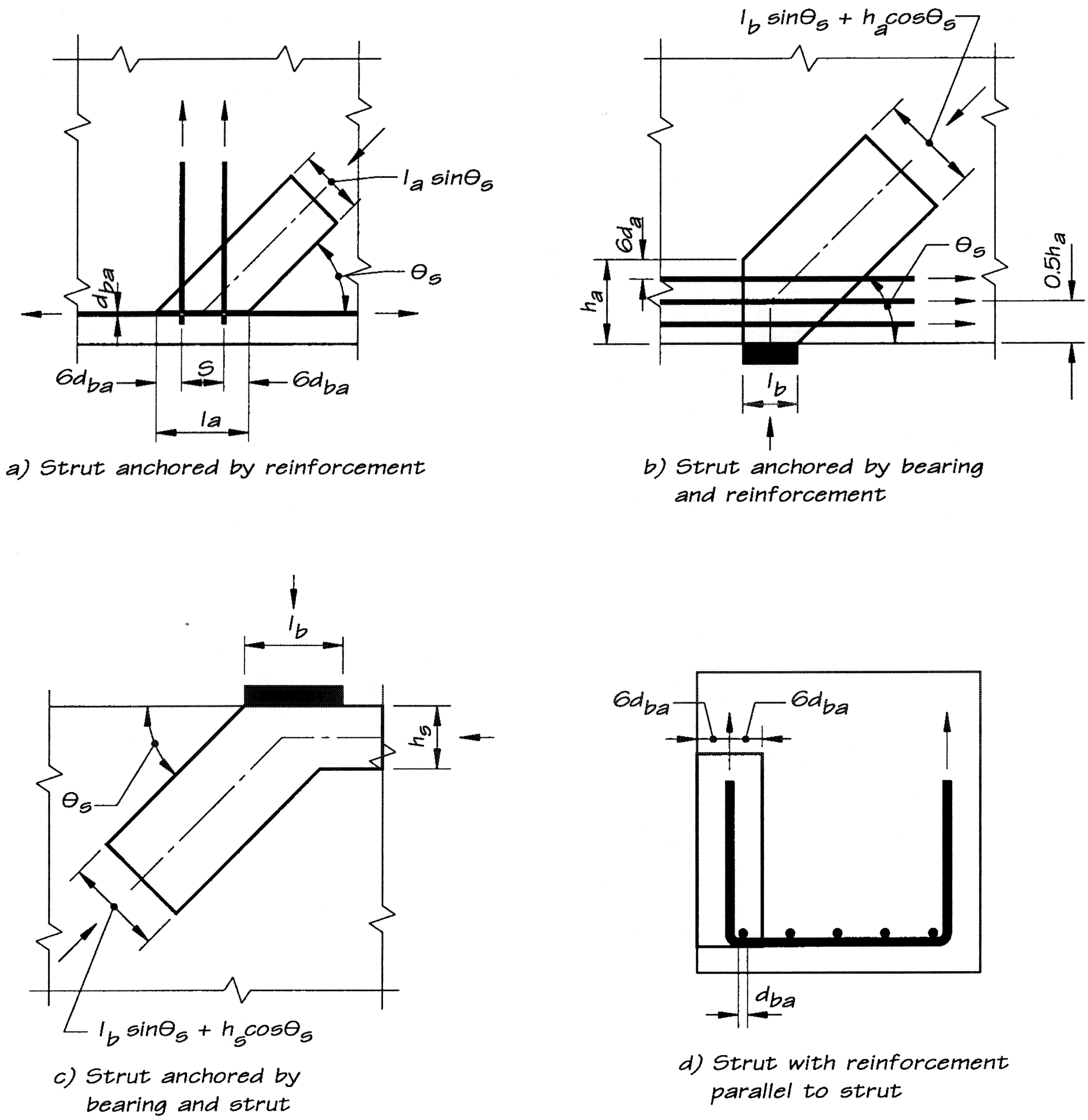
$$P_n = f_{cu} A_{cs} + f_y A_{ss}$$

where:

$A_{ss}$ = area of reinforcement in the strut ($in^2$)




Strut-and-Tie Model for Deep Beam
Figure 5.2.1-1




a) Strut anchored by reinforcement

b) Strut anchored by bearing and reinforcement

c) Strut anchored by bearing and strut

d) Strut with reinforcement parallel to strut

Influence of Anchorage Conditions on Effective Cross-Sectional Area of Strut
Figure 5.2.1-2

4. Proportioning of Tension Ties

   a. Strength of Tie

   Tension tie reinforcement shall be anchored to the nodal zones by specified embedment lengths, hooks, or mechanical anchorages. The tension force shall be developed at the inner face of the nodal zone.

   The nominal resistance of a tension tie in kip shall be taken as:

   $$P_n = f_y A_{st} + A_{ps} [f_{pe} + f_y]$$

   where:

   $A_{st}$ = total area of longitudinal mild steel reinforcement in the tie ($in^2$)
   $A_{ps}$ = area of prestressing steel ($in^2$)
   $f_y$ = yield strength of mild steel longitudinal reinforcement (ksi)
   $f_{pe}$ = stress in prestressing steel due to prestress after losses (ksi)

   b. Anchorage of Tie

   The tension tie reinforcement shall be anchored to transfer the tension force therein to the node regions of the truss in accordance with the requirements for development of reinforcement as specified in Article 5.1.2C.

5. Proportioning of Node Regions

   Unless confining reinforcement is provided and its effect is supported by analysis or experimentation, the concrete compressive stress in the node regions of the strut shall not exceed:

   - For node regions bounded by compressive struts and bearing areas: $0.85\ \varphi\ f_c'$
   - For node regions anchoring a one-direction tension tie: $0.75\ \varphi\ f_c'$
   - For node regions anchoring tension ties in more than one direction: $0.65\ \varphi\ f_c'$

   where:

   $\varphi$ = 0.7 resistance factor for bearing on concrete

   The tension tie reinforcement shall be uniformly distributed over an effective area of concrete at least equal to the tension tie force divided by the stress limits specified herein.

   In addition to satisfying strength criteria for compression struts and tension ties, the node regions shall be designed to comply with the stress and anchorage limits.

6. Crack Control Reinforcement

   Structures and components or regions thereof, except for slabs and footings, which have been designed in accordance with the provisions strut-and-tie model, shall contain an orthogonal grid of reinforcing bars near each face. The spacing of the bars in these grids shall not exceed 12.0 inches.

   The ratio of reinforcement area to gross concrete area shall not be less than 0.003 in each direction.

   Crack control reinforcement, located within the tension tie, may be considered as part of the tension tie reinforcement.

E. Shear and Torsion, ACI Method

The AASHTO Standard Specifications for Highway Bridges does not address the design of reinforced concrete members for torsion. The design for shear and torsion is based on ACI 318-95 Building Code Requirements for Structural Concrete and Commentary (318F-95) and is satisfactory for bridge members with dimensions similar to those normally used in buildings. The AASHTO LRFD Specifications Article 5.8.3.6 may also be used for design of sections subjected to shear and torsion.

F. Shear and Torsion, Strut-and-Tie Method

According to Hsu [7], utilizing ACI 318-89 for members is awkward and overly conservative when applied to large-size hollow members. Collins and Mitchell [8] propose a rational design method for shear and torsion based on the compression field theory or strut and tie method for both prestressed and non-prestressed concrete beams. These methods assume that diagonal compressive stresses can be transmitted through cracked concrete. In addition to transmitting these diagonal compressive stresses, shear stresses are transmitted from one face of the crack to the other by a combination of aggregate interlock and dowel action of the stirrups.

For recommendations and design examples for beams in shear and torsion, the designer can refer to the paper by M.P. Collins and D. Mitchell, Shear and Torsion Design of Prestressed and Non-Prestressed Concrete Beams, PCI Journal, September-October 1980, pp. 32-100 [8].

G. Deflection

Flexural members are designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength or serviceability of the structure at service load plus impact. The minimum superstructure depths are specified in AASHTO Table 8.9.2 and deflections shall be computed in accordance with Section 8.13, AASHTO *Standard Specifications for Highway Bridges*.

H. Seviceability

In addition to the deflection control requirements described above, service load stresses shall be limited to satisfy fatigue (Section 8.16.8.3) and for distribution of tension reinforcement when $f_y$ for tension reinforcement exceeds 40,000 psi (Section 8.16.8.4 AASHTO Specifications).

To control cracking of the concrete, tension reinforcement at maximum positive and negative moment sections shall be chosen so that the calculated service load stress, fs in ksi, shall be less than the value computed by:

$$f_s = \frac{z}{(d_c \times A)^{1/3}} \leq 0.6\, f_y$$

The calculated service load stress is calculated utilizing Working Stress Design (WSD) principles described below. The values of dc and A are defined in Section 8.16.8.4 of the AASHTO *Standard Specifications for Highway Bridges*. The value z shall be 130 kips per inch for girder and crossbeam reinforcing bars in negative moment regions, and all deck reinforcing bars. A value of 170 kips per inch shall be used for all other positive moment regions. Note that this check is for distribution of flexural reinforcement to control cracking. See Appendix 5.2-B2 which shows the flexural reinforcement at a pier location placed equally in top and bottom layers. When this is done, the total slab thickness can be used in computing A.

### 5.2.2 Working Stress Design Method

Prior to the strength design method, introduced in the 1973, AASHTO *Standard Specifications for Highway Bridges*, the working stress design (WSD) method was used to design bridges. Many design aids were produced as a result. The ACI Publication SP-3, *Reinforced Concrete Design Handbook Working Stress Method* [9], is a publication that was widely used by designers and several textbooks have sections devoted to WSD [1,2].

Working Stress Design principles are used to compute the tensile stress, $f_s$, and $M_{cr}$, which are used to check crack control and minimum flexural reinforcement respectively. Design aid for working stress design method for Class 3000 and 4000 concrete is provided in Appendix B4.

*P65:DP/BDM5*

## 5.3 Reinforced Concrete Box Girder Bridges

A typical box girder bridge is comprised of top and bottom concrete slabs connected by a series of vertical girder stems. This section is a guide for designing:

Top slab
Bottom slab
Girder stem (web)

For design criteria not covered, see Section 2.4.1.C.

## 5.3.1 Girder Spacing and Basic Geometries

A. Girder Spacing

The most economical web spacing for ordinary box girder bridges varies from about 8 to 12 feet. Greater girder spacing requires some increase in both top and bottom slab thickness, but the cost of the additional concrete can be offset by decreasing the total number of girder stems. Fewer girder stems reduces the amount of form work required and a lower cost.

The number of girder stems can be reduced by cantilevering the top slab beyond the exterior girders. A deck overhang of approximately one-half the girder spacing generally gives satisfactory results. This procedure usually results in a more aesthetic as well as a more economical bridge.

For girder stem spacing in excess of 12 feet or cantilever overhang in excess of 6 feet, transverse post-tensioning shall be used.

B. Basic Dimensions (Figure 5.3.1-1)

1. Top Slab Thickness, T1 (includes ½″ wearing surface)

   T1 = 12 x (S+10)/30 but not less than 7″ with overlay or 7.5″ without overlay.

2. Bottom Slab Thickness, T2

   a. Near Center Span

      T2 = 12 x ($S_{clr}$)/16 but not less than 5.5″ (normally 6.0″ is used).

   b. Near Intermediate Piers

      Thickening of the bottom slab is often used in negative moment regions to control compressive stresses that are significant.

      Transition slope = 24:1 (see T2′ in Figure 5.3.2-8).

3. Girder Stem (Web) Thickness, T3

   a. Near Center Span

      Minimum T3 = 9.0″ — vertical

      Minimum T3 = 10.0″ — if sloped

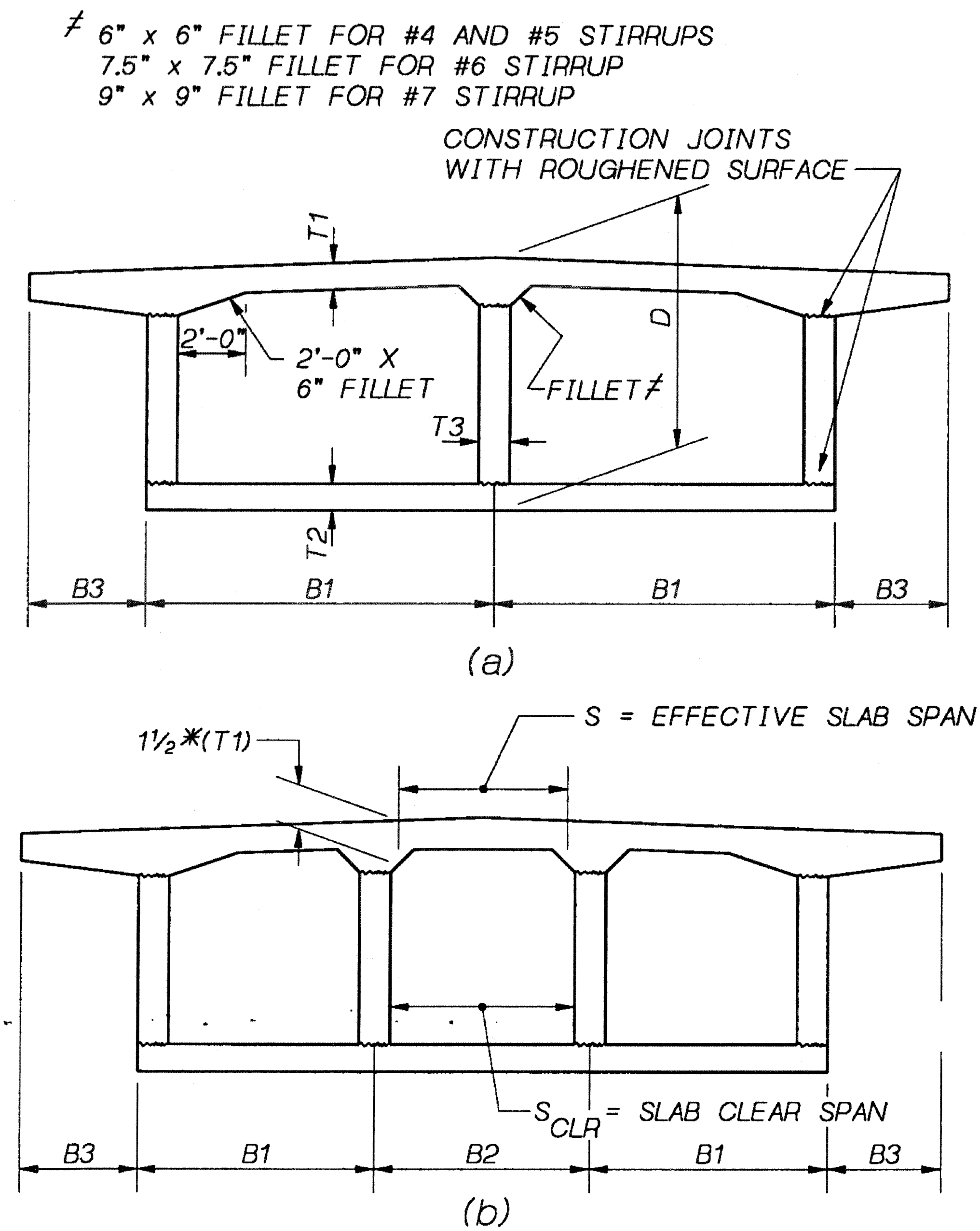
   b. Near Supports

      Thickening of girder stems is used in areas adjacent to supports to control shear requirements.

Changes in girder web thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

Maximum T3 = T3+4.0″ maximum

Transition length = 12 x (T3) in inches




Basic Dimensions
Figure 5.3.1-1

4. Intermediate Diaphragm Thickness, T4 and Diaphragm Spacing

   a. For tangent and curved bridge with R > 800 feet

      T4 = 0″ (Diaphragms are not required.)

   b. For curved bridge with R < 800 feet

      T4 = 8.0″

      Diaphragm spacing shall be as follows:

      For 600′ < R < 800′at 1/2 pt. of span.

      For 400′ < R < 600′ at 1/3 pt. of span.

      For R < 400′ at 1/4 pt. of span.

C. Construction Considerations

Review the following construction considerations to ensure that:

1. Construction joints at slab/stem interface or fillet/stem interface at top slab are appropriate.
2. All construction joints to have roughened surfaces.
3. Bottom slab is parallel to top slab (constant depth).
4. Girder stems are vertical.
5. Dead load deflection and camber to nearest 1/8″.
6. Skew and curvature effects have been considered.
7. Thermal effects have been considered.
8. The potential for falsework settlement is acceptable. This always requires added stirrup reinforcement in sloped outer webs.

D. Load Distribution

1. Unit Design

   According to the AASHTO specifications, the entire slab width shall be assumed effective for compression. It is both economical and desirable to design the entire superstructure as a unit rather than as individual girders. When a reinforced box girder bridge is designed as an individual girder with a deck overhang, the positive reinforcement is congested in the exterior cells. The unit design method permits distributing all girder reinforcement uniformly throughout the width of the structure.

2. Dead Loads

   a. Box dead loads.

   b. D.L. of top deck forms — 5 lbs. per sq. ft. of the area.<br>— 10 lbs. per sq. ft. if web spacing > 10′–0″.

   c. Traffic barrier.

   d. Overlay, intermediate diaphragm, and utility weight if applicable.

3. Live Load

   a. Superstructure

      No. of lanes = slab width (curb to curb) / 14

      Fractional lane width will be used

      For example, 58 roadway / 14 = 4.14, then no. of lanes = 4.14

   b. Substructure

      No. of lanes = slab width (curb to curb) / 12

      Fractional lane width will be ignored

      For example, 58 roadway / 12 = 4.83, then no. of lanes = 4.0

   c. Overload if applicable.

## 5.3.2 Reinforcement

This section discusses moment reinforcement for top slab, bottom slab, and intermediate diaphragms in box girders.

A. Top Slab Reinforcement

1. Near Center of Span

   Figure 5.3.2-1 shows the reinforcement required near the center of the span and Figure 5.3.2-2 shows the overhang reinforcement.

   a. Transverse reinforcing in the top and bottom layers to transfer the load to the main girder stems shall be equal in size and spacing.

   b. Bottom longitudinal "distribution reinforcement" in the middle half of the deck span ($S_{eff}$) to aid in distributing the wheel loads.

   c. Top longitudinal "temperature and shrinkage reinforcement."

2. Near Intermediate Piers

   Figure 5.3.2-3 illustrates the reinforcement requirement near intermediate piers. See Appendix 5.2-B2 for design of longitudinal deck reinforcement.

   a. Transverse reinforcing same as center of span.

   b. Longitudinal reinforcement to resist negative moment (see Figure 5.3.2-3).

   c. "Distribution of flexure reinforcement" to limit cracking (see Figure 5.3.2-3).

      Allowable $f_s = z/(d_c \text{ x } A)^{1/3} \leq 0.6fy$, where z = 130 kips per inch.

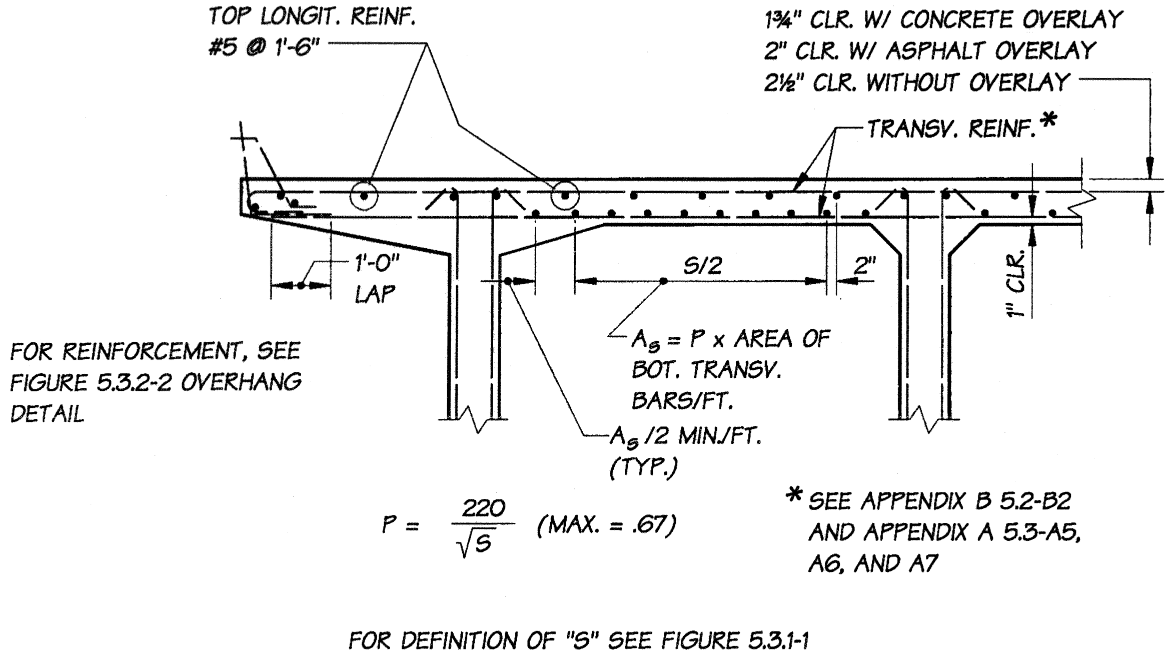
3. Bar Patterns

   a. Transverse Reinforcement

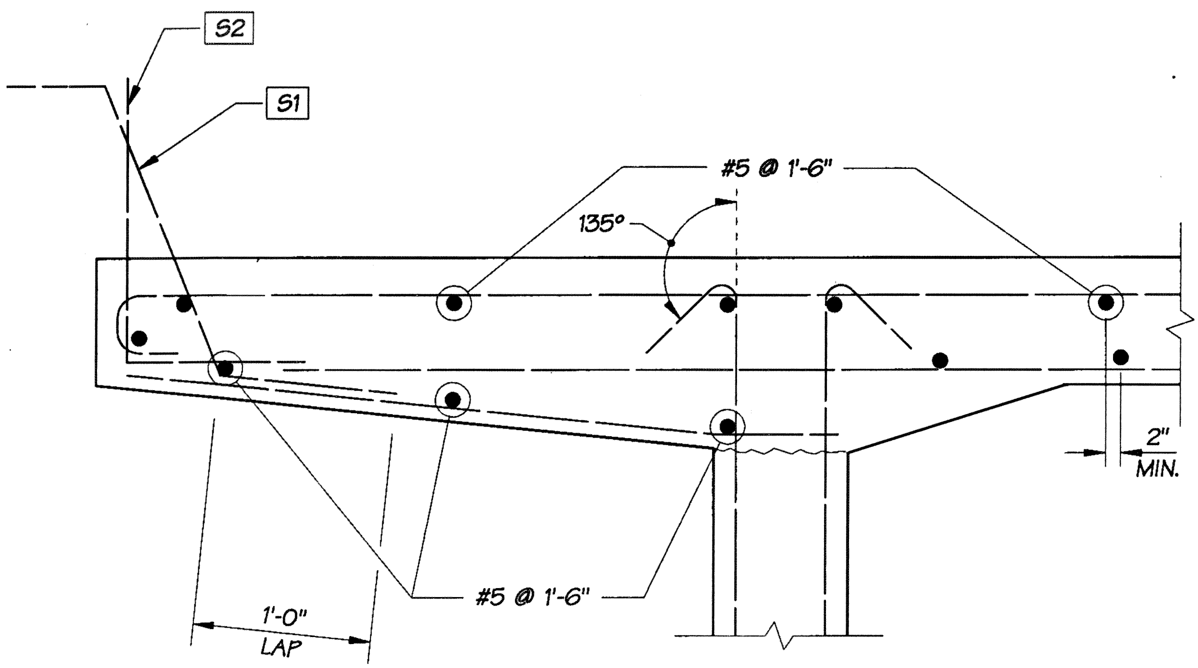
      It is preferable to place the transverse reinforcement parallel to the X-Beam and end diaphragm on skews up to 25 degrees or less (see Figure 5.3.2-4). Where skew angles exceed 25 degrees, the transverse bars are normal to bridge center line and the areas near the expansion joint and bridge ends are reinforcement by partial length bars. The bottom transverse slab reinforcement is discontinued at the X-Beam.

b. Longitudinal Reinforcement

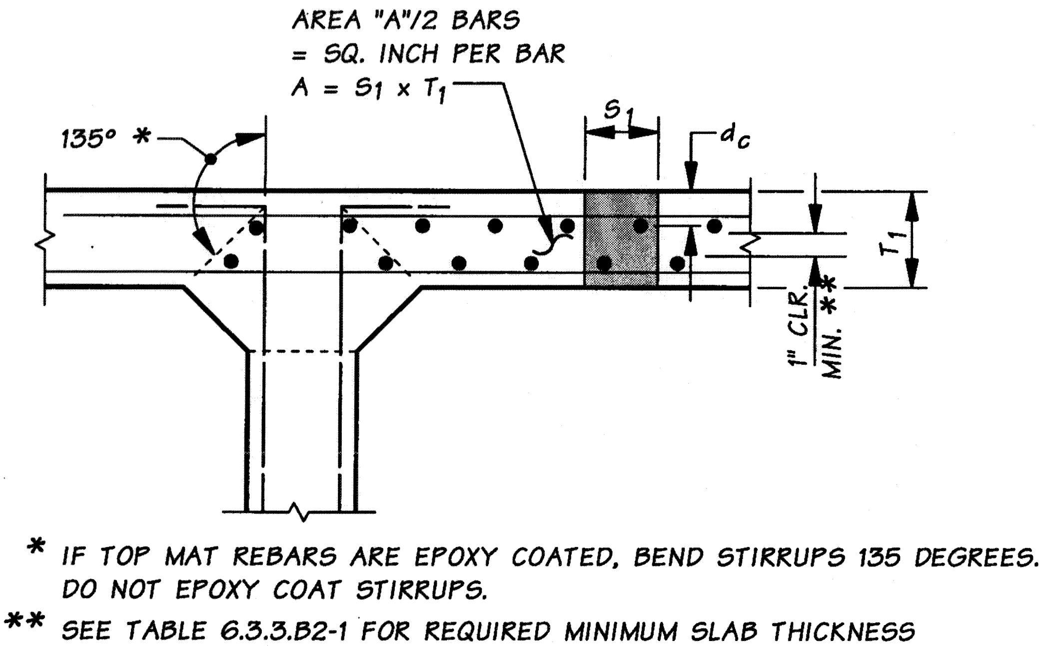
For longitudinal reinforcing bar patterns, see Chapter 6.



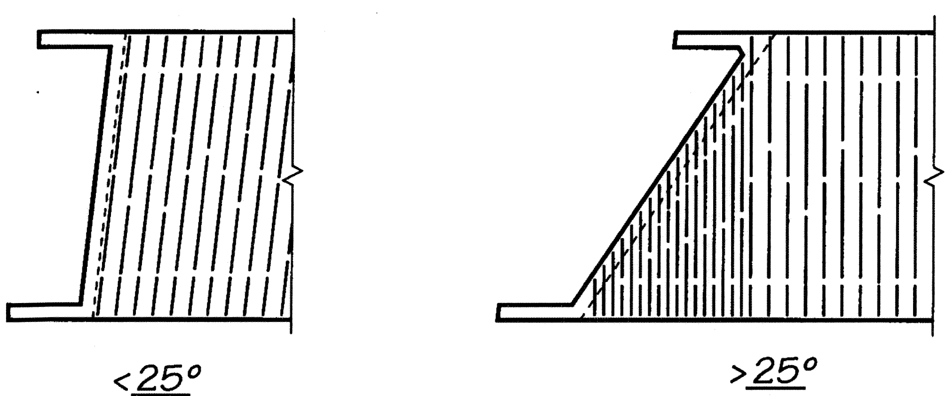

Partial Section Near Center of Span
Figure 5.3.2-1




Overhang Detail
Figure 5.3.2-2




Top Slab Flexural Reinforcing Near Intermediate Pier
Figure 5.3.2-3




Partial Plans at Abutments
Figure 5.3.2-4

B. Bottom Slab Reinforcement

1. Near Center of Span

   Figure 5.3.2-5 shows the reinforcement required near the center of the span.

   a. Minimum transverse "distributed reinforcement."

      As=0.005 x flange area with ½ $A_s$ distributed equally to each surface.

   b. Longitudinal "main reinforcement" to resist positive moment.

   c. Check "distribution of flexure reinforcement" to limit cracking (see Figure 5.3.2-5).

      Allowable $f_s = z/(d_c \times A)^{1/3} \leq 0.6fy$, where z = 170 kips per inch.

   d. Add steel for construction load (sloped outer webs).

2. Near Intermediate Piers

   Figure 5.3.2-6 shows the reinforcement required near intermediate piers.

   a. Minimum transverse reinforcement same as center of span.

   b. Minimum longitudinal "temperature and shrinkage reinforcement."

      As=0.004 x flange area with ½ $A_s$ distributed equally to each face.

   c. Add steel for construction load (sloped outer webs).

3. Bar Patterns

   a. Transverse Reinforcement

      See top slab bar patterns, Figures 5.3.2-1, 5.3.2-2, and 5.3.2-3.

      All bottom slab transverse bars shall be bent at the outside face of the exterior web. For vertical web, the tail will be 1′-0″ and for sloping exterior web 2′-0″ minimum splice with the outside web stirrups. See Figure 5.3.2-7.

   b. Longitudinal Reinforcement

      For longitudinal reinforcing bar patterns, see Chapter 6.

C. Web Reinforcement

1. Vertical Stirrups (see Figure 5.3.2-8)

   The web reinforcement should be designed for the following requirements:

   Vertical shear requirements.

   Out of plane bending on outside web due to live load on cantilever overhang.

   Horizontal shear requirements for composite flexural members.

   Minimum $\frac{A_v}{s} = 50 \frac{b_w}{fy}$ (#5 bars @ 1′-6″), where $b_w$ = no. of girder stems (T3).

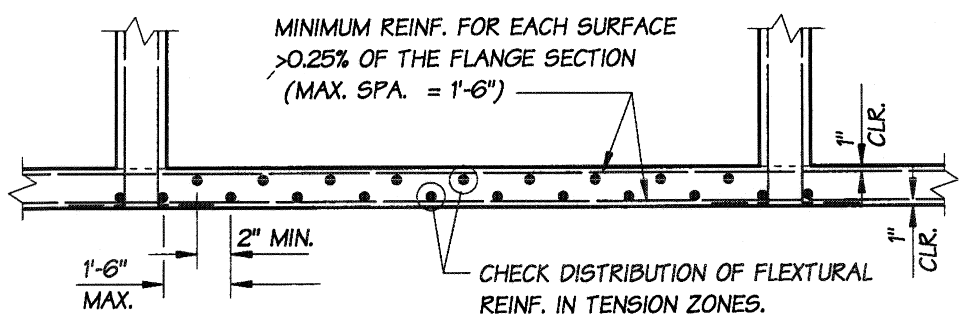
2. Web Longitudinal Reinforcement (see Figure 5.3.2-8)

   If the depth of the side face of a member exceeds 3 feet, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance d/2 nearest the flexural tension reinforcement. The area of skin reinforcement $A_{sk}$ per foot of height on each side face shall be $\geq 0.012$ (d – 30). The maximum spacing of skin reinforcement shall not exceed the lesser of d/6 and 12 inches. Such freinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one half of the flexural tensile reinforcement.

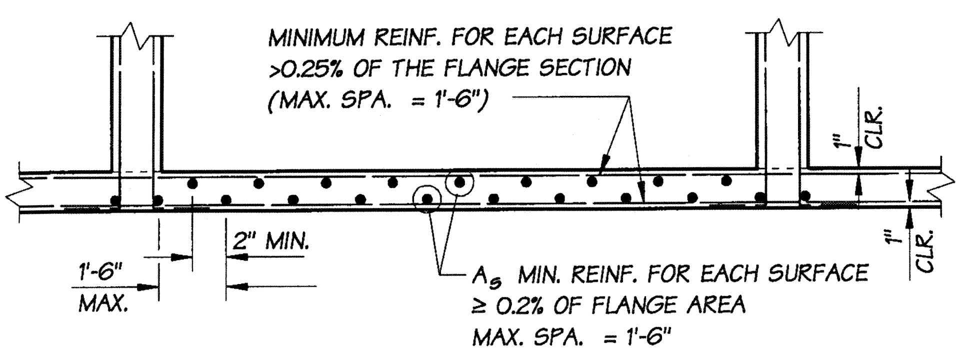
   Where As = Total required area of longitudinal reinforcing steel.

   Reinforcing steel spacing < Web Thickness (T3) or 12″.

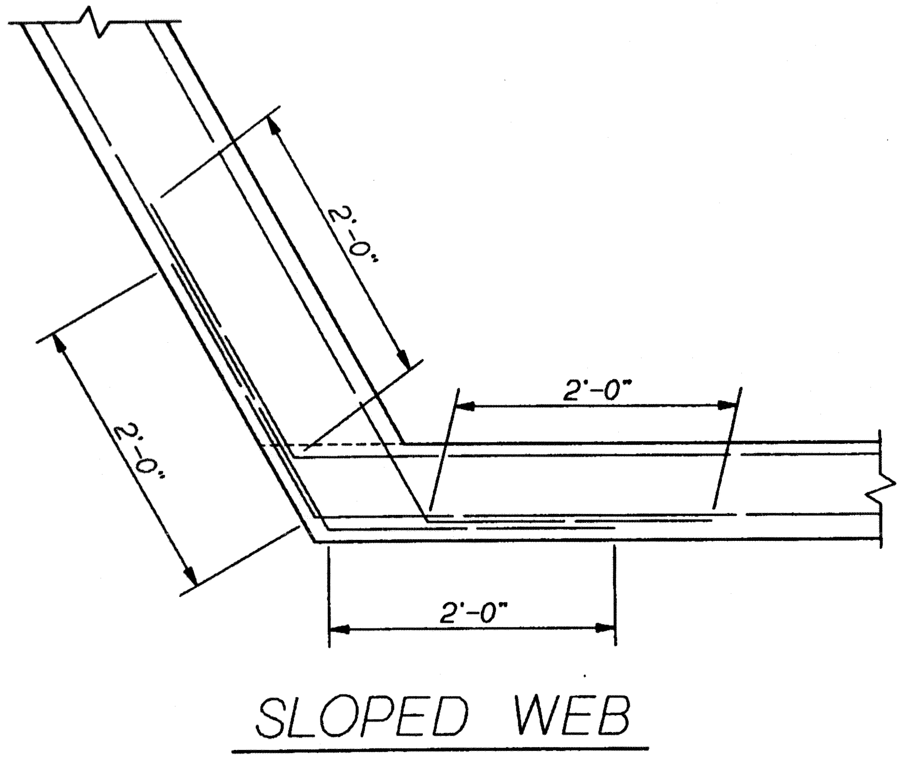
   For cast-in-place sloped outer webs, increase inside stirrup reinforcement and bottom slab top transverse reinforcement as required for the web moment locked-in during construction of the top slab. This moment about the bottom corner of the web is due to tributary load from the top slab concrete placement plus 10 psf form dead load. See Figure 5.3.2-10 for typical top slab forming.




Bottom Slab Reinforcement Near Center of Span
Figure 5.3.2-5




Bottom Slab Reinforcement Near Intermediate Pier
Figure 5.3.2-6



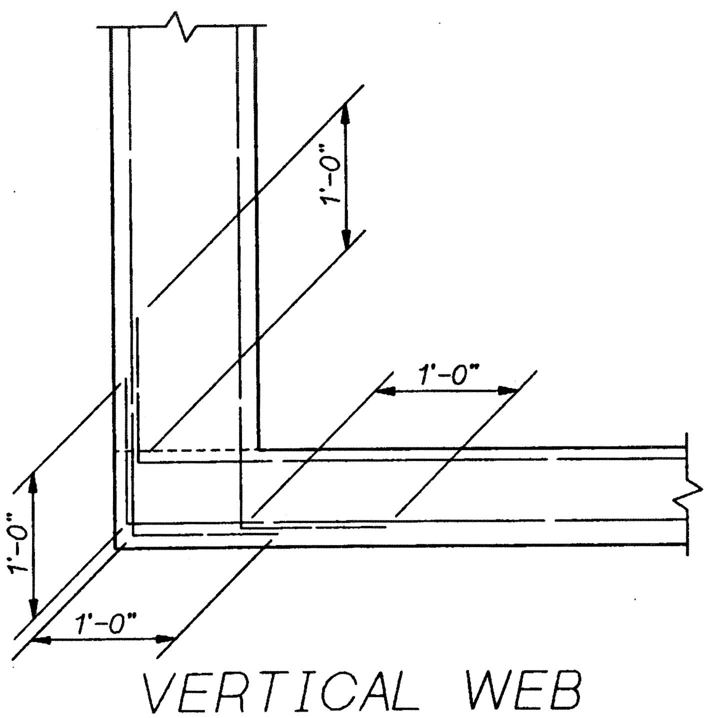
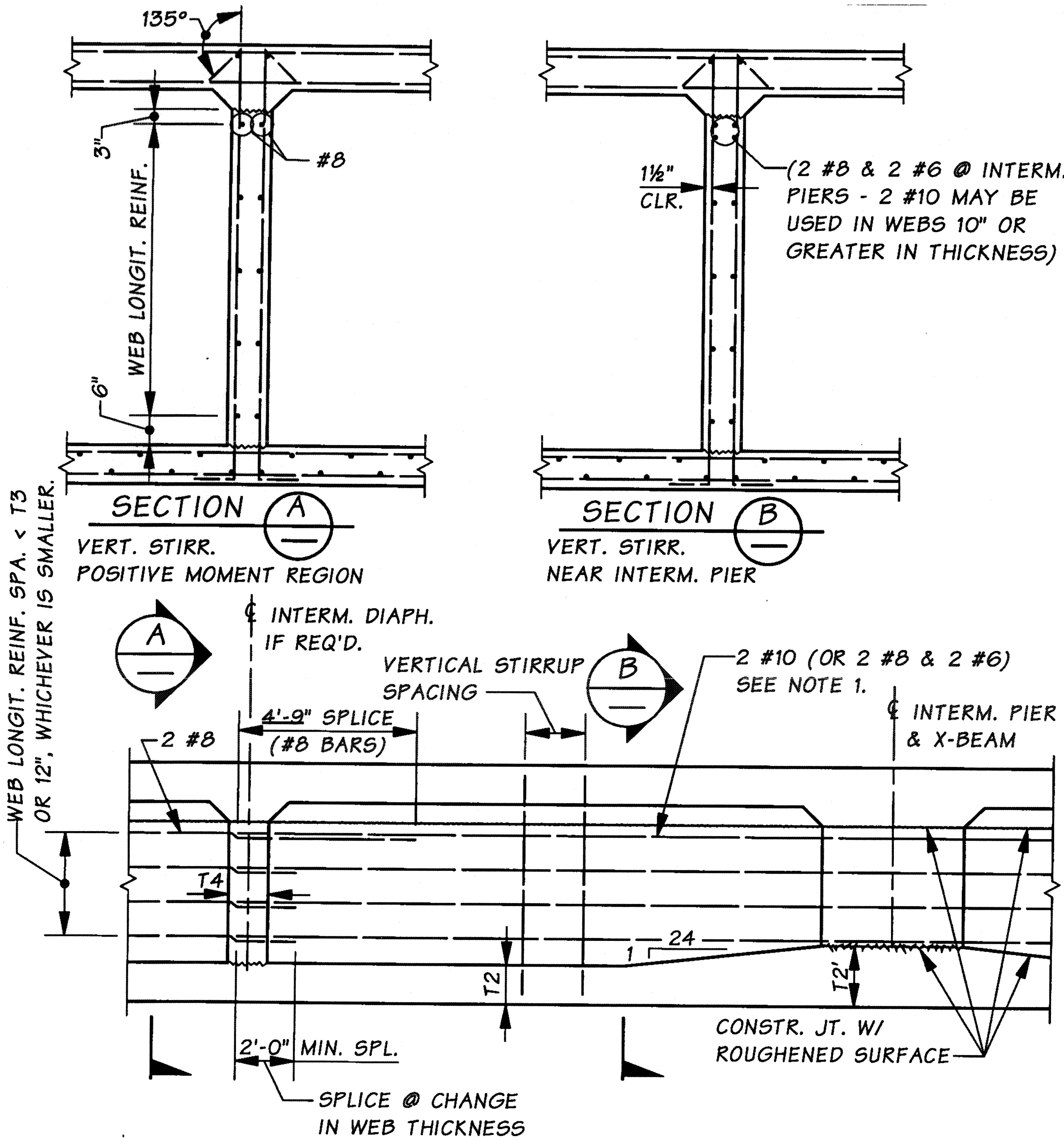




Figure 5.3.2-7




## PARTIAL LONGITUDINAL GIRDER SECTION

Use 2 #10 (2 #8 & 2 #6 for 10" or less webs) at negative moment region near piers. The length of the 2 #10 or 2 #6 bars shall be extended 35 diameters beyond the dead load point of inflecton. Do not splice the #10 (#8) bars near the pier. The #6 bars may be spliced at center pier. Use 2 #8 only for the positive moment region.

D. Intermediate Diaphragm (see Figure 5.3.2-9)

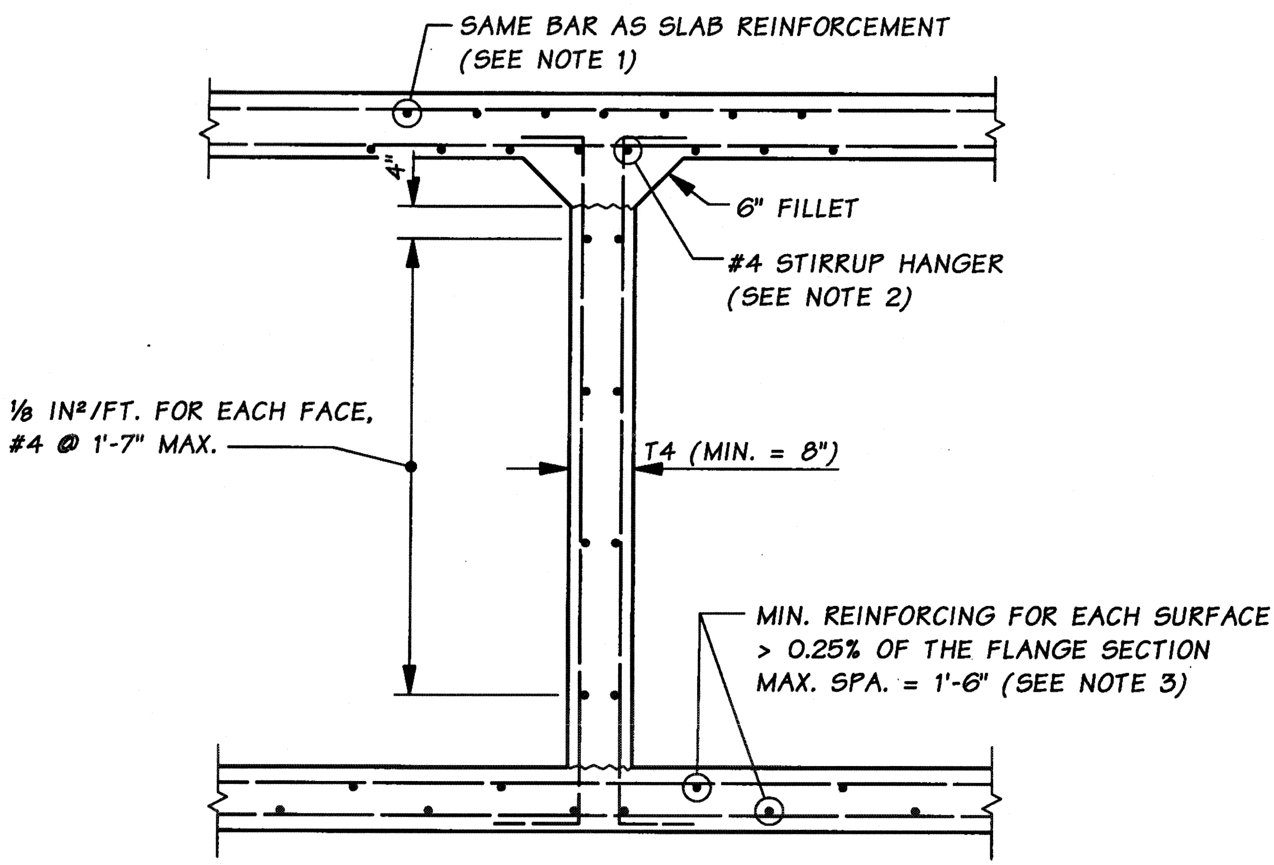
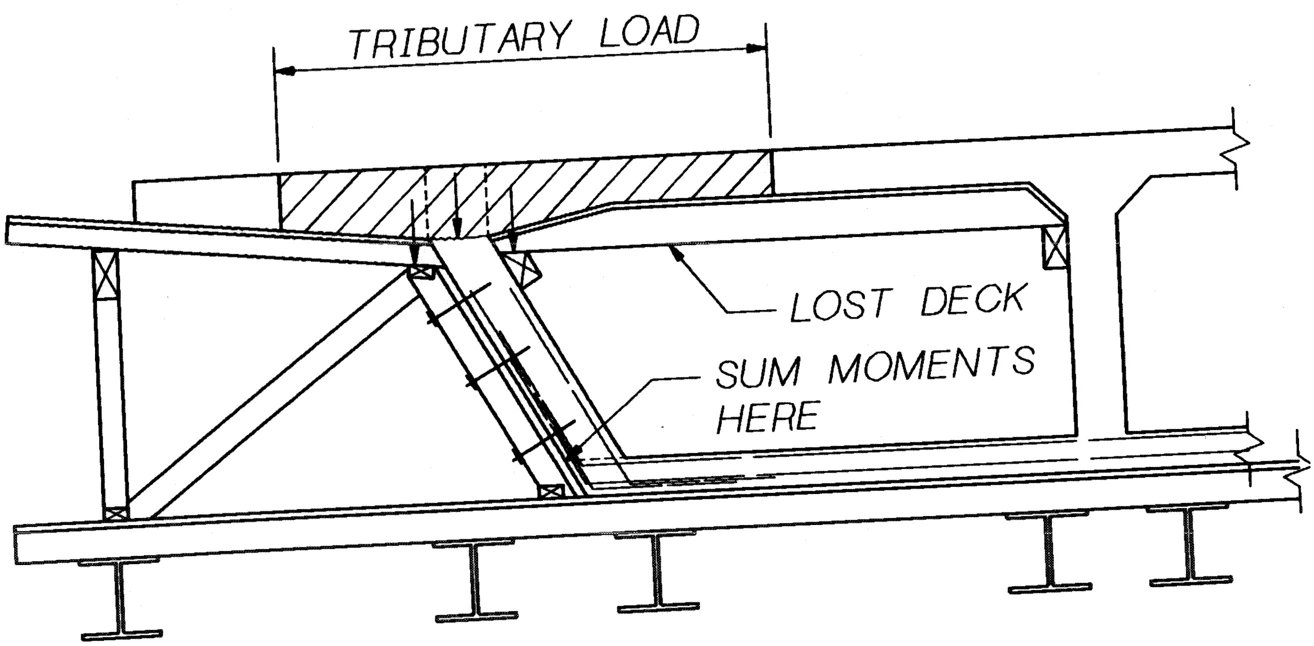



Figure 5.3.2-9

Intermediate diaphragms are not required for bridges on tangent alignment or curved bridges with an inside radius of 800 feet or greater.

Notes:

1. If the bar is not spliced, the horizontal dimension should be 4″ shorter than the slab width.
2. Stirrup hanger must be placed above longitudinal steel when diaphragm is skewed and slab reinforcement is placed normal to center of roadway. (Caution: Watch for the clearance with longitudinal steel).
3. The reinforcement should have at least one splice to facilitate proper bar placement.




Notes:

1. The diagonal brace supports web forms during web pour. After cure, the web is stiffer than the brace, and the web attracts load from subsequent concrete placements.
2. The tributary load includes half the overhang because the outer web form remains tied to and transfers load to the web which is considerably stiffer than the formwork.

Increase Web Reinf. for Locked-In Construction Load

Due to Typical Top Slab Forming for Sloped Web Box Girder
Figure 5.3.2-10

## 5.3.3 Crossbeam

A. Basic Geometry

For aesthetic purposes, it is preferable to keep the crossbeam within the superstructure so that the bottom slab of the entire bridge is a continuous plane surface interrupted only by the columns. Although the depth of the crossbeam may be limited, the width can be made as wide as necessary to satisfy design requirements. Normally, it varies from 3 feet to the depth of box but is not less than column sizes to utilize the column reinforcement (see Figure 5.3.3-1 and 5.3.3-2).

Crossbeams on box girder type of construction shall be designed as a T beam utilizing the flange in compression, assuming the deck slab acts as a flange for positive moment and bottom slab a flange for negative moment. The effective overhang of the flange on a cantilever beam shall be limited to six times the flange thickness.

The bottom slab thickness is frequently increased near the crossbeam in order to keep the main box girder compressive stresses to a desirable level for negative girder moments (see Figure 5.3.2-8). This bottom slab flare also helps resist negative crossbeam moments. Consideration should be given to flaring the bottom slab at the crossbeam for designing the cap even if it is not required for resisting main girder moments.

B. Reinforcing Steel Details

Special attention should be given to the details to ensure that the column and crossbeam reinforcement will not interfere with each other. This can be a problem especially when round columns with a great number of vertical bars must be meshed with a considerable amount of positive crossbeam reinforcement passing over the columns.

1. Top Reinforcement

Provide negative moment reinforcement at the ¼ point of the square or equivalent square columns (see Appendix 5.3-A1 and 5.3-A4).

a. When Skew Angle < 25 Degrees

If the bridge is tangent or slightly skewed and the deck reinforcement is parallel to the cross beam, the negative cap reinforcement can be placed either in contact with top deck negative reinforcement or directly under the main deck reinforcement (see Figure 5.3.3-1). Reinforcement must be epoxy coated if the location of reinforcement is less than 4″ below top of deck.

b. When Skewed Angle > 25 Degrees

When the structure is on a greater skew and the deck steel is normal or radial to the longitudinal centerline of the bridge, the negative cap reinforcement should be lowered to below the main deck reinforcement (see Figure 5.3.3-2).

c. To avoid cracking of concrete, interim reinforcements are required below the construction joint in diaphgragms and crossbeams.

The interim reinforcements shall develop a moment capacity of 1.2 Mcr where Mcr may be given as:

$$M_{cr} = \frac{fr\ Ig}{y_t}$$

$$fr = 7.5\sqrt{f_c'}$$

$$M_{cr} = 1.25\, bh^2 \sqrt{f_c'}$$

$$M_n = 1.2M_{cr} = 1.5\, bh^2 \sqrt{f_c'}$$

$$A_s = \frac{0.85\ f_c'\ b}{f_y}\left(d - \sqrt{d^2 - \frac{31.3725M_n}{f_c'\underline{b}}}\right)$$

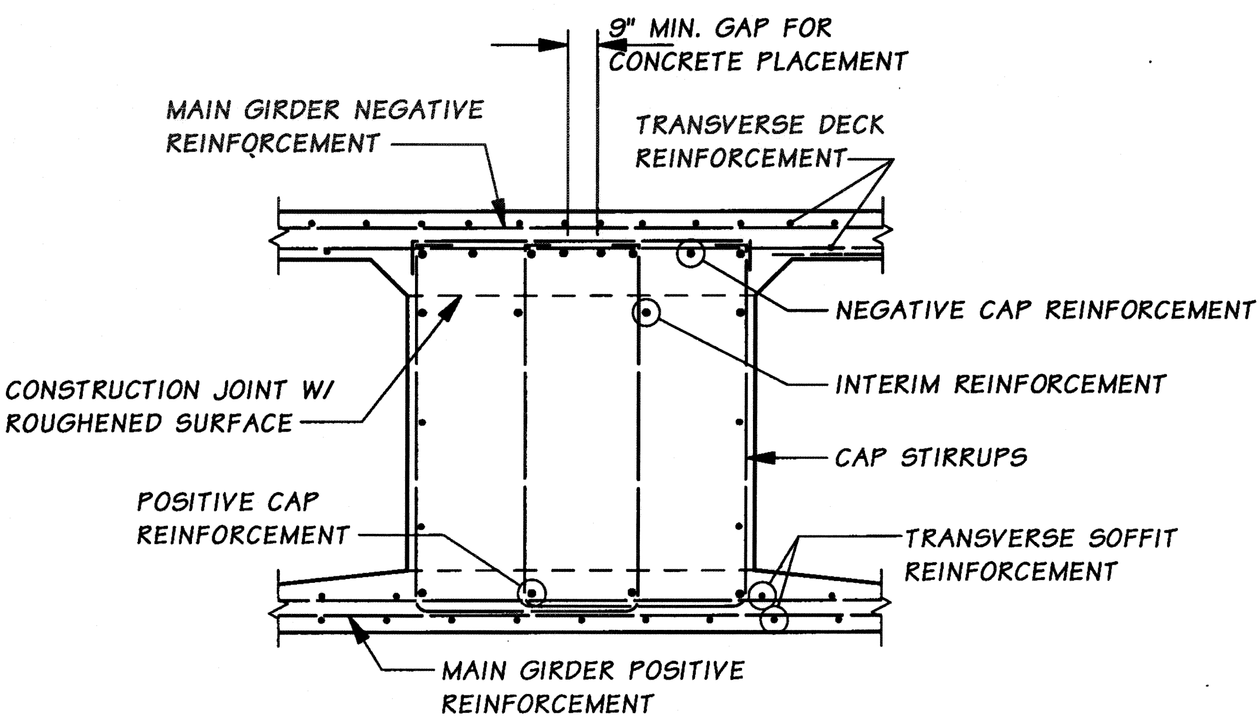
### 5.3.4 End Diaphragm

A. Basic Geometry

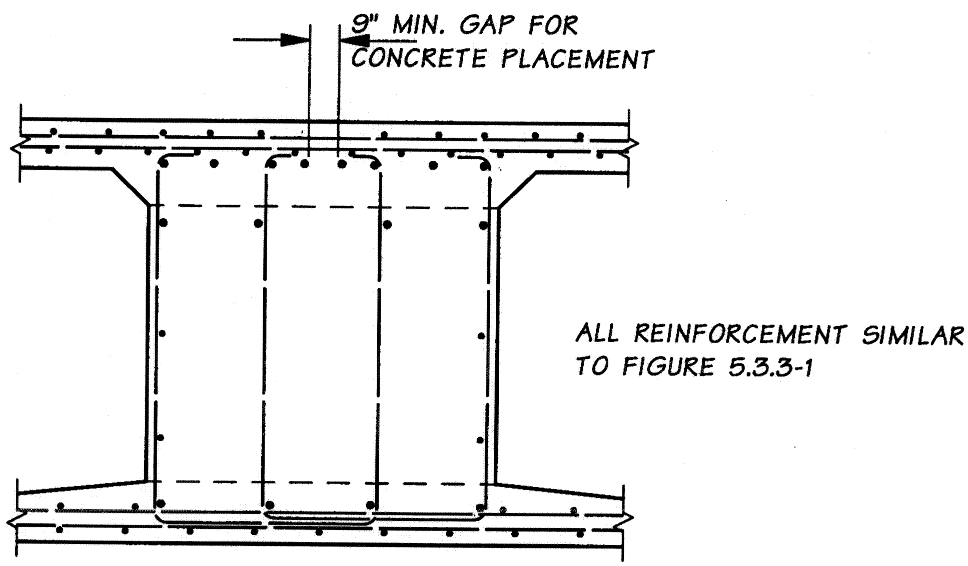
Bearings at the end diaphragms are usually located under the girder stems and transfer loads directly to the pier (see Figure 5.3.4-1). In this case, the diaphragm width should be equal to or greater than bearing sole plate grout pads (see Figure 5.3.4-2).

Designer should provide access space for maintenance and inspection of bearings.

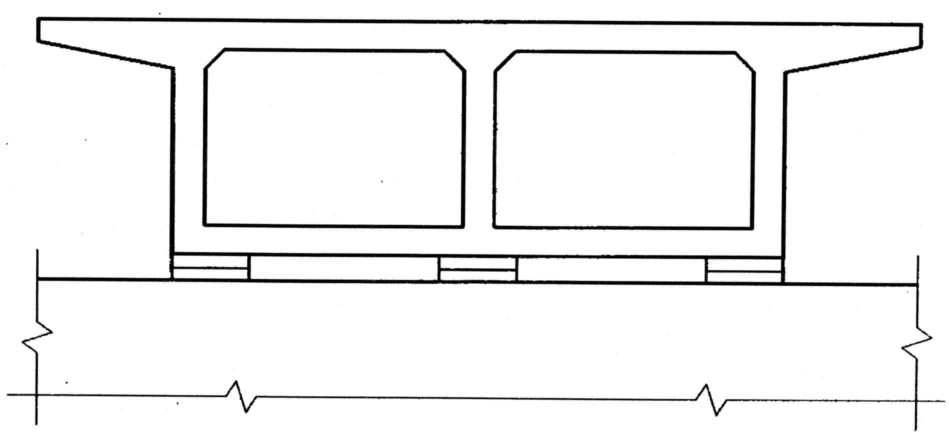
Allowance should be provided to remove and replace the bearings. Lift point locations, jack capacity, number of jacks, and maximum permitted lift should be shown in the plan details.



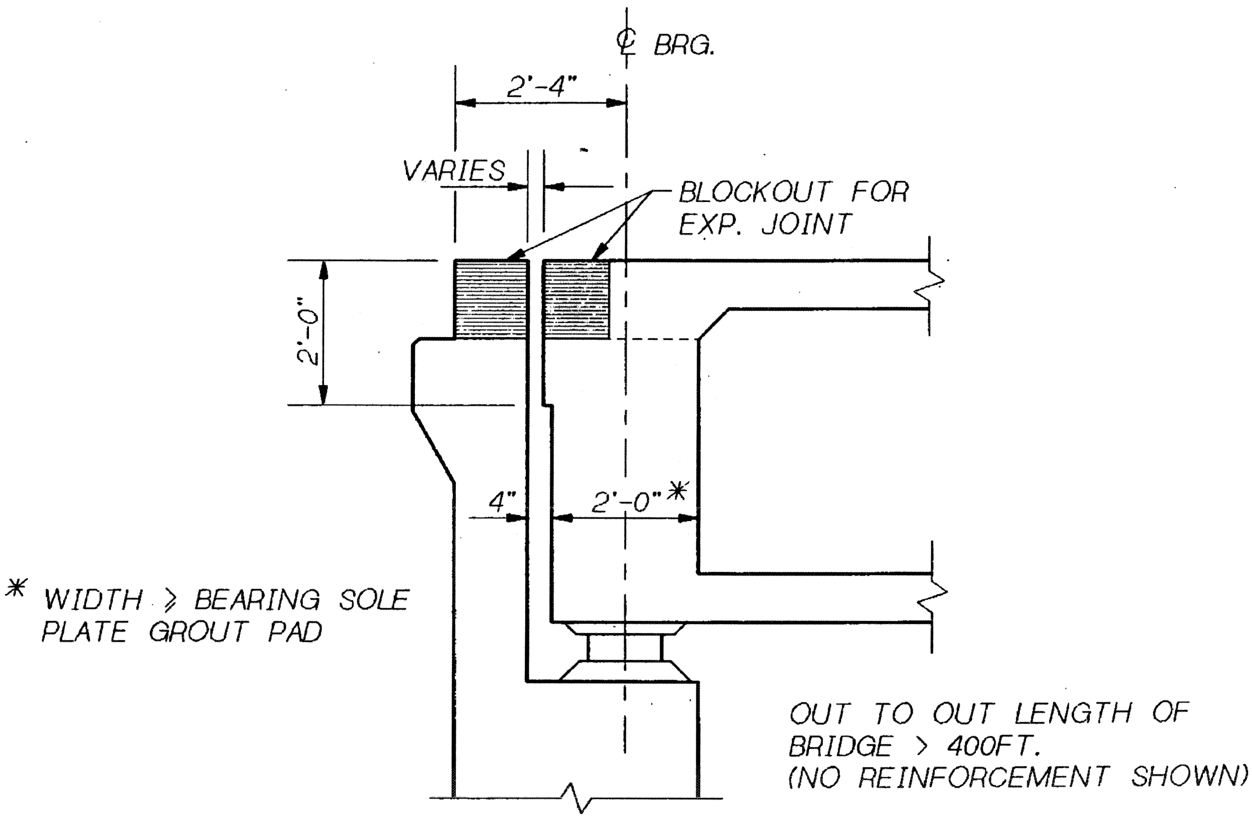

Skew Angle ≤ 25°
Crossbeam Top Reinforcement
Figure 5.3.3-1




Skew Angle > 25°
Crossbeam Top Reinforcement
Figure 5.3.3-2



Bearing Locations, Lift Points, Jack Capacity, and Maximum Lift Permitted at End Diaphragm
Figure 5.3.4-1




"L" Abutment End Diaphragm
Figure 5.3.4-2

The end diaphragms should be wide enough to provide adequate reinforcing embedment length. When the structure is on a skew greater than 10 degrees and the deck steel is normal or radial to the center of the bridge, the width should be enough to accommodate the embedment length of the reinforcement.

The most commonly used type of end diaphragm is shown in Figure 5.3.4-3. The dimensions shown here are used as a guideline and should be modified if necessary. This end diaphragm is used with a stub abutment and overhangs the stub abutment. It is used on bridges with an overall or out-to-out length less than 400 feet. If the overall length exceeds 400 feet, an "L" abutment should be used.

B. Reinforcing Steel Details

Typical reinforcement details for an end diaphragm are shown in Figure 5.3.4-4.

## 5.3.5 Dead Load Deflection and Camber

Camber is the adjustment made to the vertical alignment to compensate for the anticipated dead load deflection and the long-term deflection caused by shrinkage and creep. The multipliers for estimating long-term deflection and camber for reinforced concrete flexural members may be taken as shown in Table 1.

Multipliers for Estimating Long-term Deflection and Camber of Concrete Members
Table 5.3.5-1

| | Multiplier Coefficient |
|---|---|
| **Girder Adjacent to Existing/Stage Construction** | |
| Deflection (downward) — apply to the elastic deflection due to the weight of member | 1.90 |
| Deflection (downward) — apply to the elastic deflection due to superimposed dead load only | 2.20 |
| **Girder Away From Existing/Stage Construction** | |
| Deflection (downward) — apply to the elastic deflection due to the weight of member | 2.70 |
| Deflection (downward) — apply to the elastic deflection due to superimposed dead load only | 3.00 |

In addition to dead load deflection, forms and falsework tend to settle and compress under the weight of freshly placed concrete. The amount of this takeup is dependent upon the type and design of the falsework, workmanship, type and quality of materials and support conditions. The camber should be modified to account for anticipated takeup in the falsework.

### 5.3.6 Thermal Effects

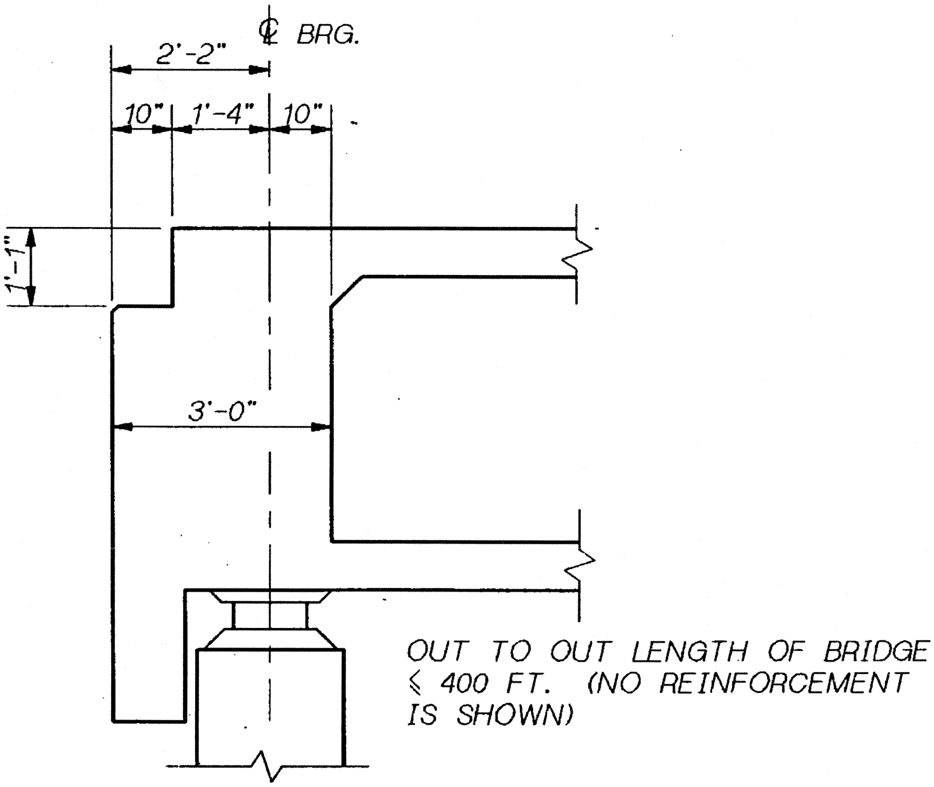
Concrete box girder bridges are subjected to stresses and/or movements resulting from temperature variation. Temperature effects result from time-dependent variations in the effective bridge temperature and from temperature differentials within the bridge superstructure.

A. Effective Bridge Temperature and Movement

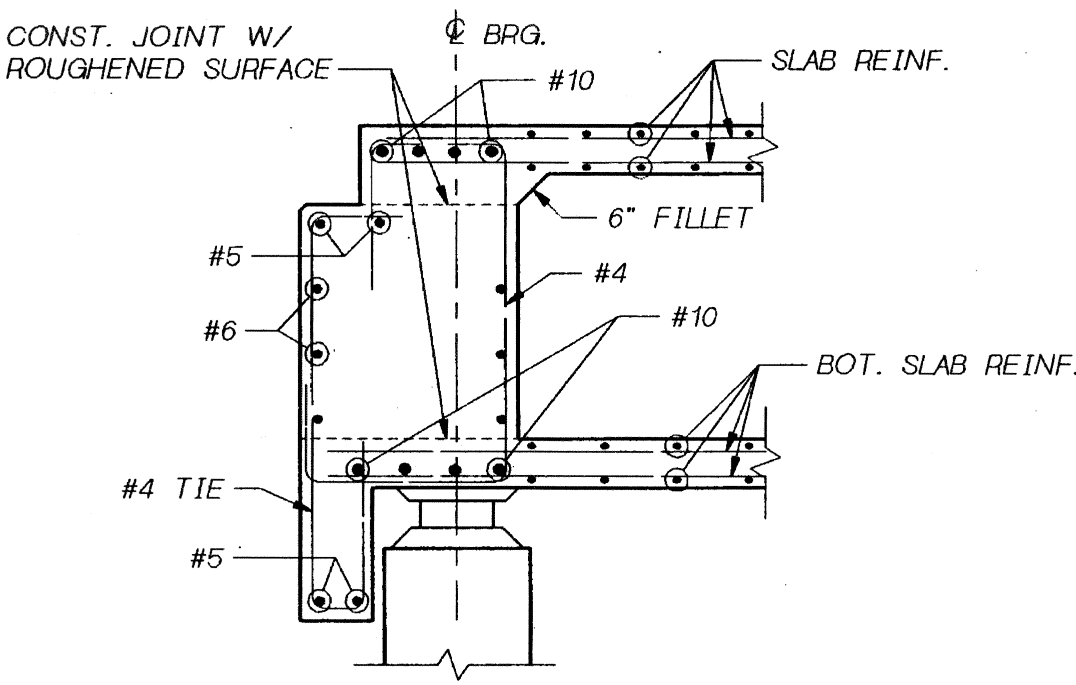
Fluctuation in effective bridge temperature causes expansion and contraction of the structure. Proper temperature expansion provisions are essential in order to ensure that the structure will not be damaged by thermal movements. These movements, in turn, induce stresses in supporting elements such as columns or piers, and result in horizontal movement of the expansion joints and bearings. For more details, see Chapter 8.

B. Differential Temperature

Although time-dependent variations in the effective temperature have caused problems in both reinforced and prestressed concrete bridges, detrimental effects caused by temperature differential within the superstructure have occurred only in prestressed bridges. Therefore, computation of stresses and movements resulting from the vertical temperature gradients is not included in this chapter. For more details, see AASHTO Guide Specifications, Thermal Effects on Concrete Bridge Superstructures (1989).




End Diaphragm With Stub Abutment
Figure 5.3.4-3




Typical End Diaphragm Reinforcement
Figure 5.3.4-4

## 5.3.7 Hinges

Hinges are one of the weakest links of box girder bridges subject to earthquake forces and it is desirable to eliminate hinges or reduce the number of hinges. For more details on the design of hinges, see Section 5.4.

Designer should provide access space or pockets for maintenance and inspection of bearings.

Allowance should be provided to remove and replace the bearings. Lift point locations, maximum lift permitted, jack capacity, and number of jacks should be shown in the hinge plan details.

## 5.3.8 Utility Openings

A. Confined Spaces

A confined space is any place having a limited means of exit which is subject to the accumulation of toxic or flammable contaminants or an oxygen deficient environment. Confined spaces include but are not limited to pontoons, box girder bridges, storage tanks, ventilation or exhaust ducts, utility vaults, tunnels, pipelines, and open-topped spaces more than 4 feet in depth such as pits, tubes, vaults, and vessels. The designer should provide for the following:

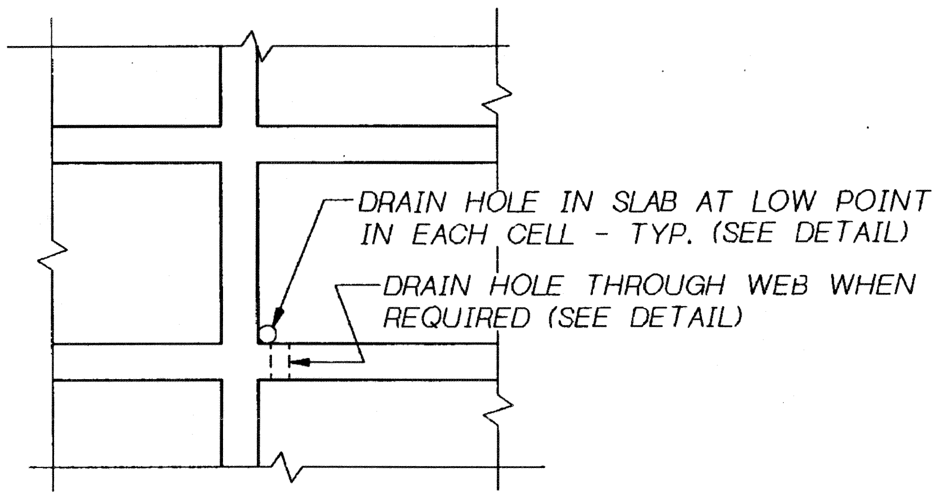
- A sign with "Confined Space Authorized Personnel Only."
- In the "Special Provisions Check List," alert and/or indicate that a special provision might be needed to cover confined spaces.

B. Drain Holes

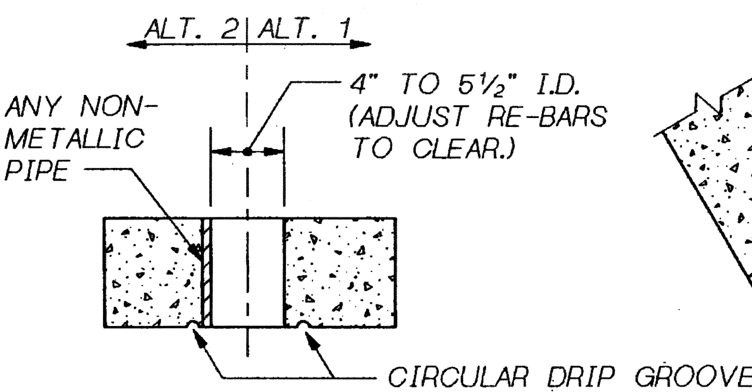
Drain holes should be placed in the bottom slab at the low point of each cell to drain curing water during construction and any rain water that leaks through the deck slab. Additional drains shall be provided as a safeguard against water accumulation in the cell (especially when waterlines are carried by the bridge). In some instances, drainage through the bottom slab is difficult and other means shall be provided (i.e., cells over large piers and where a sloping exterior web intersects a vertical web). In this case, a horizontal drain should be provided through the vertical web. Figure 5.3.8-1 shows drainage details for the bottom slab of concrete box girder bridges.

C. Access Hole and Air Vent Holes

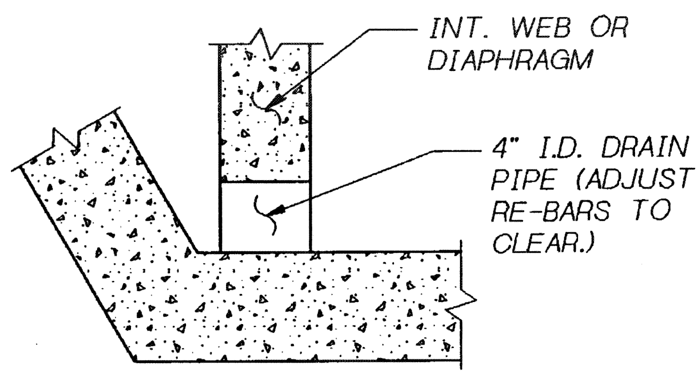
Access holes with doors should be placed in the bottom slab if necessary to inspect utilities inside cells (i.e., waterline, conduits, E.Q. restrainers, etc.). Figure 5.3.8-2 and 5.3.8-3 shows access hole and air vent hole details. Air vents are required when access holes are used.




DRAIN HOLES
SHOWN ON FRAMING PLAN



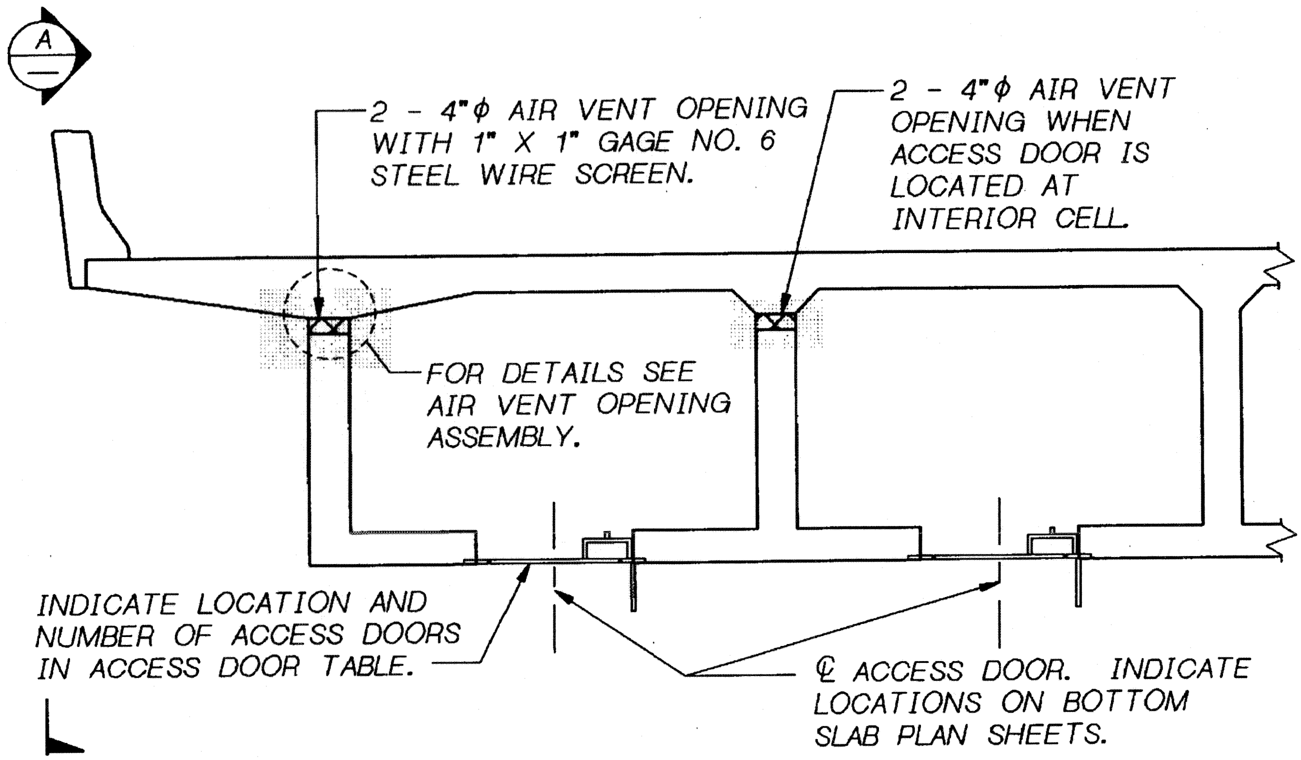

BOTTOM SLAB
DRAIN HOLE DETAIL



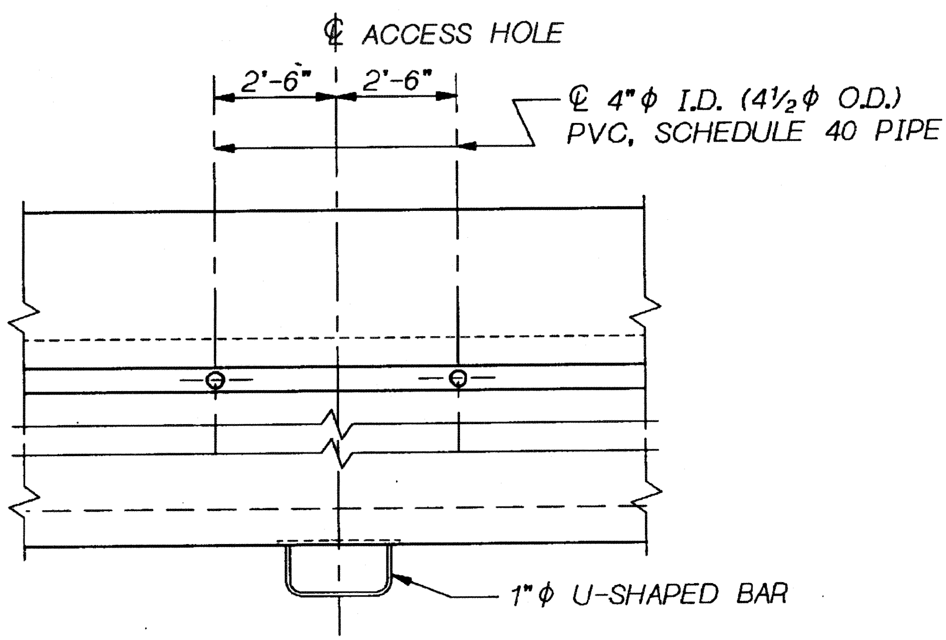

WEB DRAIN
HOLE DETAIL

Figure 5.3.8-1

P65:DP/BDM5




ELEVATION - AIR VENT HOLE IN WEBS




VIEW A

Figure 5.3.8-2

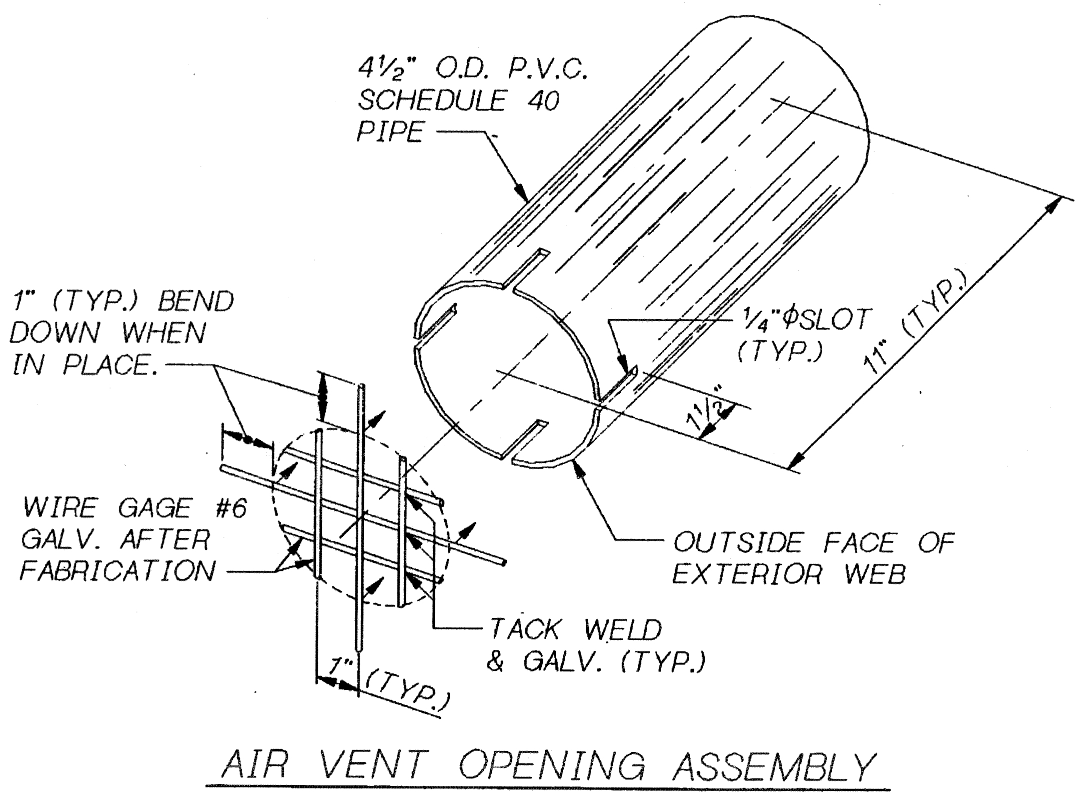

4½" O.D. P.V.C.
SCHEDULE 40
PIPE
1" (TYP.) BEND
DOWN WHEN
IN PLACE.
¼"ϕSLOT
(TYP.)
11" (TYP.)
1½"
WIRE GAGE #6
GALV. AFTER
FABRICATION
OUTSIDE FACE OF
EXTERIOR WEB
TACK WELD
& GALV. (TYP.)
1" (TYP.)
AIR VENT OPENING ASSEMBLY

Figure 5.3.8-3

## 5.4 Hinges and Inverted T-Beam Pier Caps

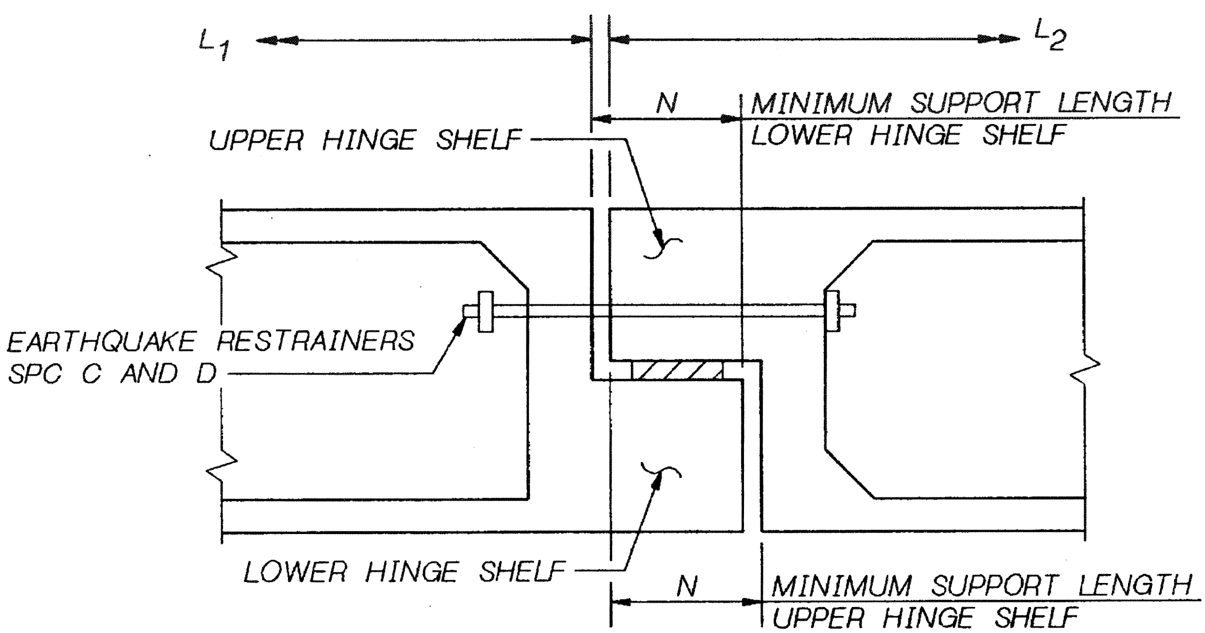
Hinges and inverted T-beam pier caps require special design and detailing considerations. Continuous hinge shelves (both top and bottom projecting shelves) and continuous ledges of inverted T-beam pier caps, which support girders, are shown in Figures 5.4-1 and 5.4-2 respectively. In each case, vertical tensile forces (hanger tension) act at the intersection of the web and the horizontal hinge shelf or ledge. In the ledges of inverted T-beam pier caps, passage of live loads may also cause reversing torsional stresses which together with conventional longitudinal shear and bending produce complex stress distributions in the ledges [10,11].

Provide minimum shelf or ledge support lengths (N, N1, and N2) and provide positive longitudinal linkage (e.g., earthquake restrainers) [12] in accordance with the current AASHTO seismic design requirements.

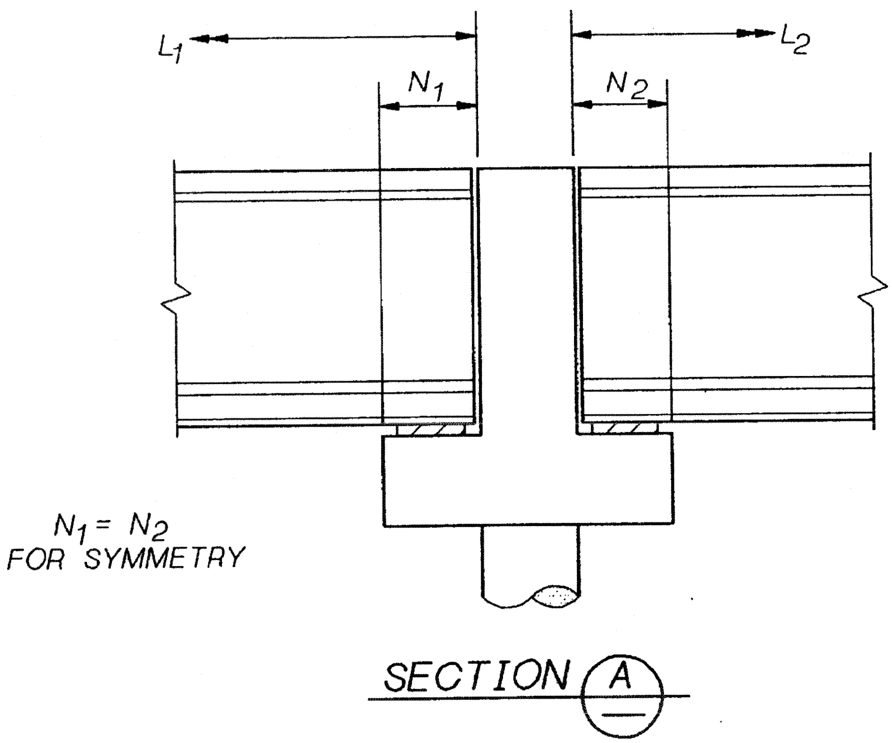
A. Local Failure Modes

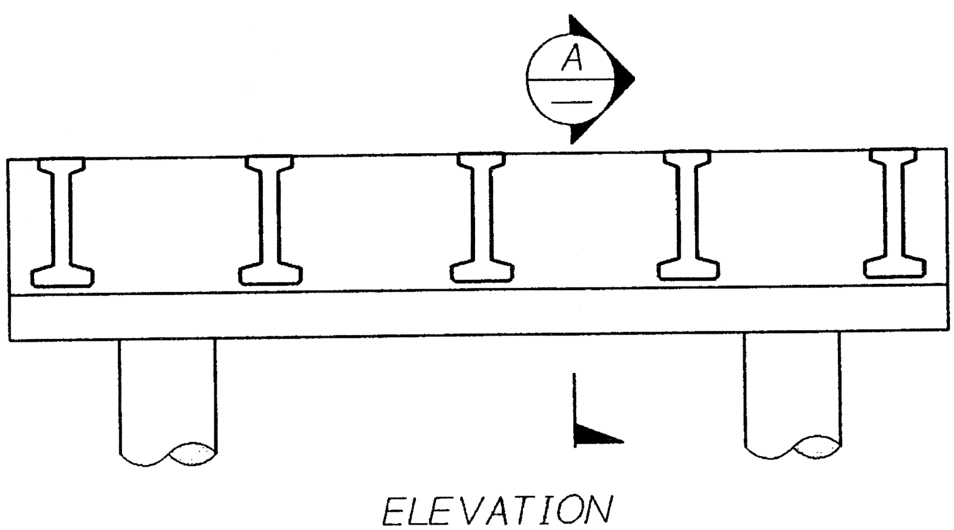
In addition to conventional longitudinal bending and shearing forces, there are several local modes of failure which should be addressed in the design [10,11]. These are: shear friction failure, flexural failure, hanger tension failure, punching shear failure of the horizontal hinge shelf or ledge, and spalling under the bearing.

Figure 5.4-3 shows these local failure modes and potential cracks. For all conditions, except for the bearing strength check, use $\phi$=0.85. For the bearing strength check, use $\phi$=0.7 [13].




Continuous Hinge
Figure 5.4-1



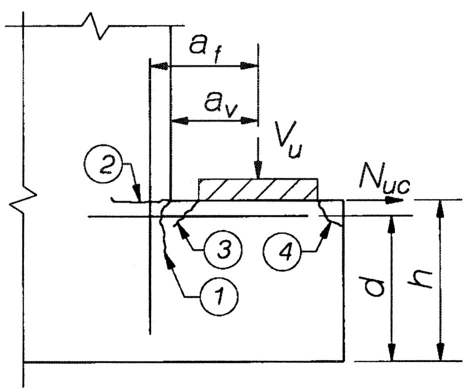

Inverted T-Beam Pier Cap
Figure 5.4-2

The forces acting on the hinge shown in Figure 5.4-3 are: shear, Vu; horizontal tensile force, Nuc; and moment, Mu.

| | | | |
|---|---|---|---|
| $V_u$ | = | Factored Shear (Dead Load + Live Load + Impact) | (1) |
| $N_{uc}$ | ≥ | $0.2V_u$, but less than $1.0V_u$ | (2) |
| $M_u$ | = | $V_u(af) + N_{uc}(h\text{-}d)$ | (3) |
| where: af | = | Flexural moment arm is the distance from the reaction to the centerline of the hanger reinforcement, and shall include the thermal movement of the reaction, $V_u$. | |
| h-d | = | Moment arm for the horizontal load, $N_{uc}$. | |

The horizontal tensile load, $N_{uc}$, is due to indeterminate causes such as restrained shrinkage or temperature stresses and is considered a live load [13].

In addition, service load conditions should also be checked for deflections and crack control.




| | |
|---|---|
| Crack 1 | could lead to a flexural or shear friction failure mode. |
| Crack 2 | necessitates hanger reinforcement. |
| Crack 3 | could lead to a punching shear failure. |
| Crack 4 | can be avoided by reducing the bearing stress or allowing more edge distance. |

Failure Modes and Potential Cracks
Figure 5.4-3

B. Shear Friction Design

1. Interior Bearing

Figure 5.4-4 shows the effective shelf width used to compute the allowable shear strength. The ratio av/d shall satisfy equation (4) and the factored shear force (including shelf dead load) shall satisfy both equations (5) and (6) [13]:

$$a_v/d \leq 1.0 \qquad (4)$$

$$V_u \leq \phi\,(0.2f_c')(W+4a_v)(d) \qquad (5)$$

$$V_u \leq \phi\mu\,(A_{vf})(fy) \qquad (6)$$

where:
- $a_v$ = Distance from the reaction to the vertical face
- $d$ = Depth from compression face to tensile reinforcement
- $\phi$ = 0.85
- $0.2f_c' \leq$ 800 psi
- $W+4a_v$ = Effective shelf width
- $\mu$ = 1.4 for cast-in-place concrete (e.g., monolithic construction, no construction joint)
- $A_{vf}$ = Shear friction reinforcement

When $W+4a_v > S$, check:

$$V_u \leq \phi\,(0.2f_c')(S)(d) \qquad (7)$$

2. Bearing at End of Hinge or Ledge

When $S > 2c < (W+4a_v)$, check:

$$Vu \leq \phi\,(0.2f_c')(2c)(d) \qquad (8)$$

When $S > (W+4a_v) < 2c$, check:

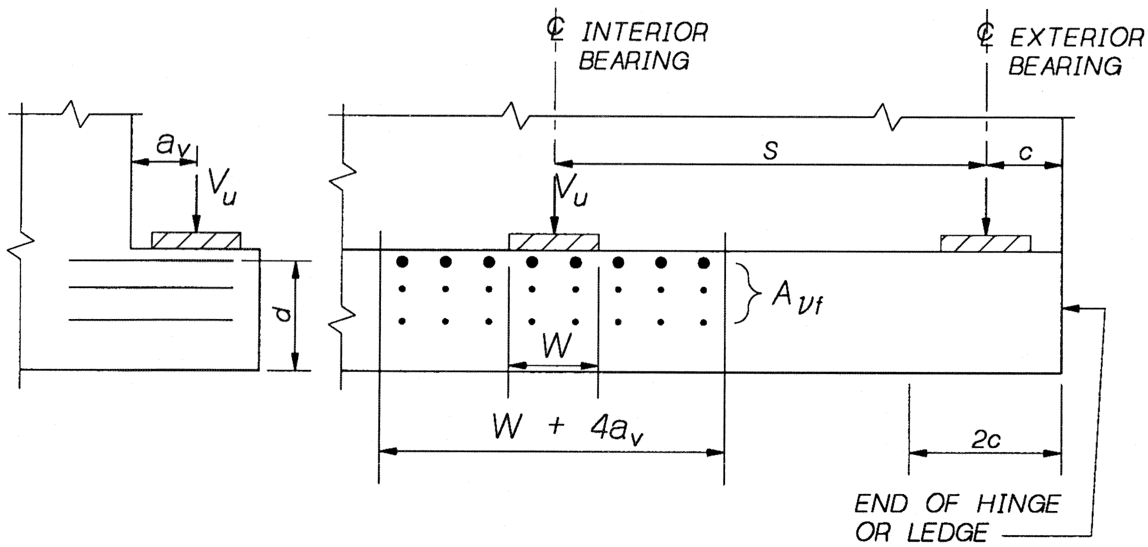
$$V_u \leq \phi\,(0.2f_c')(W+4a_v)(d) \qquad (9)$$

When $(W+4a_v) > S > 2c$, check:

$$V_u \leq \phi\,(0.2f_c')(S)(d) \qquad (10)$$

In addition, equation (6) shall be satisfied. $A_{vf}$ is distributed over 2c, $W+4a_v$, or S, whichever is less.

where
- c = Distance from the end of the hinge or ledge to the center of the exterior bearing.
- S = Center-to-center of girders or hinge seat bearings.




Shear Friction Design
Figure 5.4-4

C. Flexural Design (Figure 5.4-5)

The primary reinforcement, As, for the shelf or ledge shall be determined from equations (11), (12), and (13), whichever is greater [13]:

$$A_s \geq A_f + A_n \qquad (11)$$

$$A_s \geq 2(A_{vf})/3 + A_n \qquad (12)$$

$$A_s \geq \rho min\ (W+5a_f)(d) \qquad (13)$$

where:

$\rho min = 0.04(f_c'/fy)$

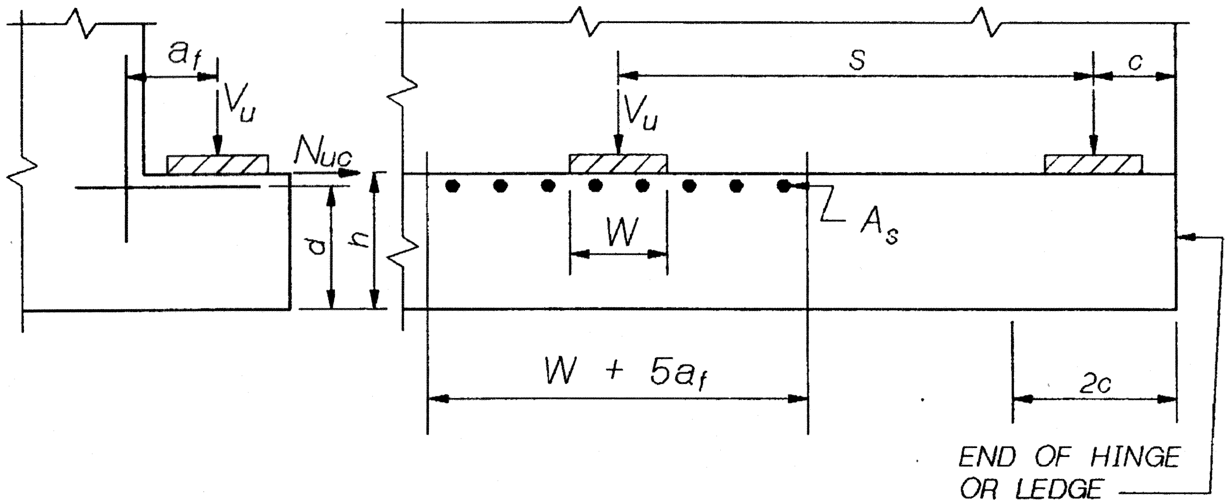
$A_f$ = Flexural reinforcement required for $M_u$

$A_{vf}$ = Shear friction reinforcement

$A_n$ = Tensile reinforcement = $N_{uc}/\phi(fy)$

In addition, closed stirrups or ties parallel to As with a total area Ah of not less than 0.5(As-An) shall be uniformly distributed within two thirds of the effective depth adjacent to As [13].

If the effective width $W+5a_f \geq S$ place the reinforcement over distance S. At the ends of the hinge or ledge, distribute the reinforcement over distance 2c, S, or $W+5a_f$, whichever is less.




Flexural Design
Figure 5.4-5

D. Hanger Tension Design (Figure 5.4-6)

The hanger tension reinforcement, Ahr, shall satisfy both of the following strength and serviceability equations:

$V_u \leq \phi A_{hr}/s)(fy)(S)$ Strength (14)

$V \leq (A_{hr}/s)(0.5fy)(W+3a_v)$ Serviceability (15)

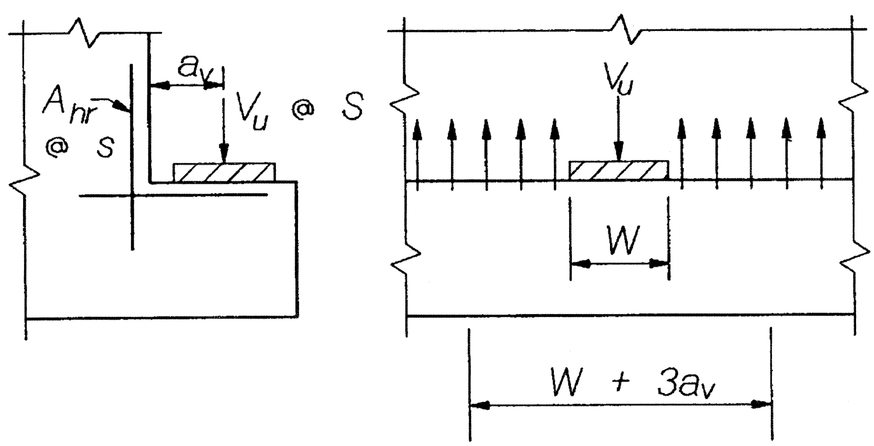
where:

$A_{hr}$ = Hanger reinforcement in square inches

s = Spacing of the hanger reinforcement

V = Service load reaction

$W+3a_v$ = Effective width for hanger reinforcement-Serviceability




Hinge Hanger Reinforcement
Figure 5.4-6

In addition to equations (14) and (15), the following equation shall also be satisfied for inverted T-beam pier caps (see Figure 5.4-7):

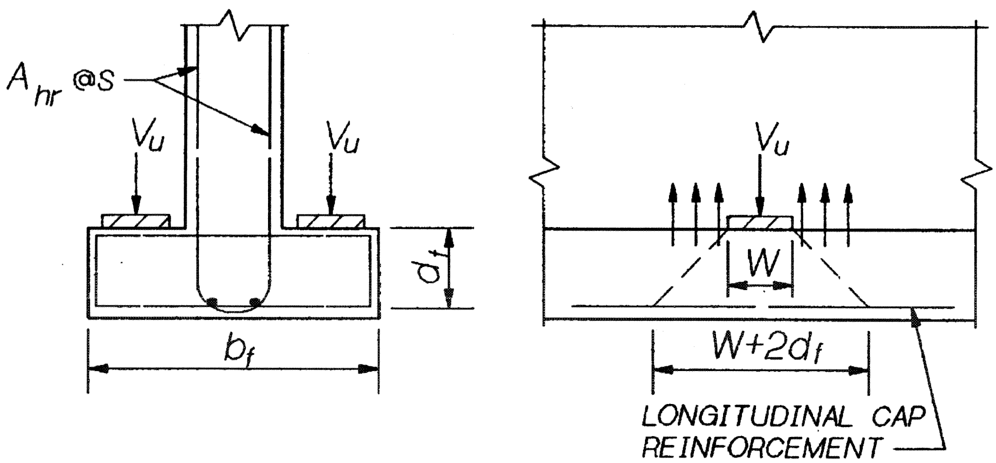
$$2V_u \leq 2[2\phi\sqrt{f_c'}\, b_f d_f] + \phi(a_{hr}/s)(fy)(W+2d_f) \qquad (16)$$

where $b_f$ = Width of bottom flange of inverted T-beam

$d_f$ = Distance from top of ledge to center of longitudinal cap reinforcement near the bottom flange of the inverted T-beam

$W+2d_f$ = Effective width for hanger reinforcement for inverted T-beam.

If $S>(W+2d_f)$, it is not necessary to add the stirrup reinforcement for conventional shear and torsion to the hanger reinforcement. Ensure that the stirrup reinforcement satisfies either the conventional longitudinal shear and torsion reinforcement requirements or the hanger reinforcement requirement, whichever is greater. If $S<(W+2d_f)$, it will be necessary to add the required hanger reinforcement to that required for shear and torsion [11].




Inverted T-Beam Hanger Reinforcement
Figure 5.4-7

E. Punching Shear Check

As shown in Figure 5.4-8, punching shear of the horizontal shelves of hinges and ledges of inverted T-beam pier caps should be checked. For an interior bearing, check:

$$V_u \leq \phi(4\sqrt{f_c'})(W + 2L' + 2d)(d) \qquad (17)$$

For an exterior bearing at the end of a hinge or inverted T-beam cap, check:

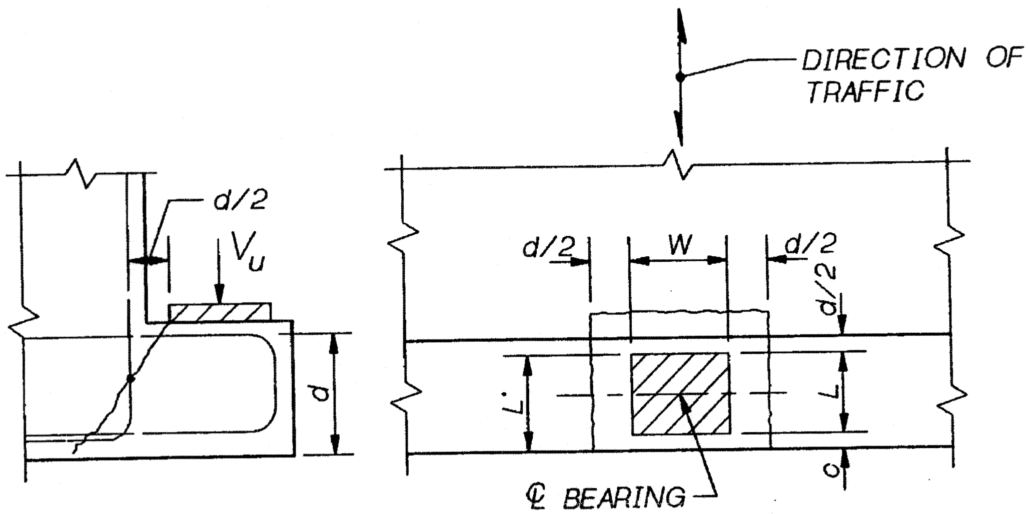
$$V_u \leq \phi(4\sqrt{f_c'})(W + L' + d)(d) \qquad (18)$$

where:

$4\sqrt{f_c'}$ = Allowable tensile strength of concrete for punching shear

W = Width of the rectangular bearing perpendicular to the longitudinal axis of the bridge (e.g., width parallel to the centerline of bearings)

L′ = Length from face of hinge or ledge to back of bearing = L+c




Punching Shear at Interior Bearing
Figure 5.4-8

F. Bearing Strength Check

To prevent spalling under the bearing, the bearing stress should not exceed $0.85(\phi)(f_c')$ [13]:

$$V_u \leq 0.85(\phi)(f_c')(W)(L) \quad (19)$$

where: $\phi$ = 0.70

L = Length of the rectangular bearing parallel to the longitudinal axis of the bridge (e.g., parallel to the direction of traffic).

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## 5.5 Widenings

This section provides general guidance for the design of bridge widenings. Included are additions to the substructure and the superstructure of reinforced concrete box girder, flat slab, T-beam, and precast-prestressed girder bridges. For additional information, see ACI Committee Report, Guide for Widening Highway Bridges [15].

### 5.5.1 Review of Existing Structures

A. General

Obtain the following documents from existing records for preliminary review, design, and plan preparation:

1. Reduced copy of "As-Built"contract plans from our microfilm records in Bridge Records, Office of Bridges and Structures.
2. Reduced copy of original contract plans and special provisions, which can be obtained from Engineering Records (Plans Vault), Records Control. These will not include the "As-Built" plans, since they are made prior to receiving the "As-Built" plans from the Project Engineer. Backup microfilm records are also maintained by Engineering Records (Plans Vault), Records Control, but the "As-Built" plans may not be current.
3. Check with the Bridge Preservation Unit for records of any unusual movements/rotations and other structural information.
4. Original design calculations, which are stored in State Archives and can be retrieved by Bridge Records personnel.
5. Current field data on Supplemental Site Data Form (including current deck elevations at interface of widening and existing deck, as well as cross slopes), are obtained from Region. Current field measurements of existing pier crossbeam locations are recommended so that new prestressed girders are not fabricated too short or too long. This is particularly important if piers have been constructed with different skews. This information may not be available in any existing plans, so field trips may be necessary to determine actual details.
6. Original and current Foundation Reports from the Materials Lab or from the Plans Vault.
7. Change Order files to the original bridge contract in Records Control Unit.

B. Original Contract Plans and Special Provisions

Location and size of reinforcement, member sizes and geometry, location of construction joints, details, allowable design soil pressure, and test hole data are given on the plans. Original contract plans can be more legible than the microfilm copies.

The special provisions may include pertinent information that is not covered on the plans or in the Standard Specifications.

C. Original Calculations

The original calculations should be reviewed for any "special assumptions" or office criteria used in the original design. The actual stresses in the structural members, which will be affected by the widening, should be reviewed. This may affect the structure type selected for the widening.

D. Final Records

For major widening/renovation projects, the Final Records should be reviewed particularly for information about the existing foundations and piles. Sometimes the piles indicated on the original plans were omitted, revised, or required preboring. Final Records are available from Records Control or Bridge Records (Final Records on some older bridges may be in storage at the Materials Lab).

## 5.5.2 Analysis and Design Criteria

A. General

Each widening represents a unique situation and construction operations may vary between widening projects. The guidelines in this section are based on over 20 years of WSDOT design experience with bridge widenings.

1. Appearance

The widening of a structure should be accomplished in such a manner that the existing structure does not look "added on to." When this is not possible, consideration should be given to enclosure walls, cover panels, paint, or other aesthetic treatments. Where possible and appropriate, the structure's appearance should be improved by the widening.

2. Materials

Preferably, materials used in the construction of the widening shall have the same thermal and elastic properties as the materials used in the construction of the original structure.

3. Load Distribution and Construction Sequence

The members of the widening should be proportioned to provide similar longitudinal and transverse load distribution characteristics as the existing structure. Normally this can be achieved by using the same cross sections and member lengths that were used in the existing structure.

The construction sequence and degree of interaction between the widening and the existing structure, after completion, shall be fully considered in determining the distribution of the dead load for design of the widening and stress checks for the existing structure. The distribution of live load shall be in accordance with the AASHTO specifications. Where precast-prestressed girders are used to widen an existing cast-in-place concrete box girder or T-beam bridge, the live load distribution factor for interior girder(s) shall be S/5.5.

The construction sequence or stage construction should be clearly shown in the plans to avoid confusion and misinterpretation during construction. A typical construction sequence may involve placing the deck concrete, removing the falsework, placing the concrete for the closure strip, and placing the concrete for the traffic barrier. Indicate in the plans a suggested stage construction plan to avoid misinterpretation.

4. Specifications

The design of the widening shall conform to the current AASHTO Specifications and the state of Washington's *Standard Specifications for Road, Bridge, and Municipal Construction.*

The method of design for the widening shall be by load factor design methods even though the original design may have been by service load design.

5. Geometrical Constraints

   The overall appearance and geometrical dimensions of the superstructure and columns of the widening should be the same or as close as possible to those of the existing structure. This is to ensure that the widening will have the same appearance and similar structural stiffness as the original structure.

6. Strength of Concrete

   The allowable stresses shown in the latest AASHTO Specifications are to be used. For concrete structures located in rural areas or where the volume of concrete is less than 30 cubic yards, use Class 4000 ($f_c' = 4000$ psi) and Grade 60 reinforcement. For projects located in urban areas and having a volume of concrete greater than 30 cubic yards, Class 5000 may be specified only if necessary to meet structural requirements and if facilities are available. Concrete with a greater strength may be used, if needed, with consultation and approval of the Bridge Design Engineer.

7. Overlay

   It should be established at the preliminary plan stage if an overlay is required as part of the widening.

8. Strength of the Existing Structure

   A review of the strength of the main members of the existing structure shall be made for construction conditions utilizing AASHTO Load Factors.

   A check of the existing main members after attachment of the widening shall be made for the final design loading condition.

   If the existing structural elements do not have adequate strength, consult your supervisor or in the case of consultants, contact the Consultant Liason Engineer for appropriate guidance.

   If significant demolition is required on the existing bridge, consideration should be given to requesting concrete strength testing for the existing bridge and including this information in the contract documents.

9. Special Considerations

   a. For structures that were originally designed for HS20 loading, HS25 shall be used to design the widening. For structures that were originally designed for less than HS20, consideration should be given to replacing the structure instead of widening it.

   b. Where large cambers are expected, a longitudinal joint between the existing structure and the widening may be considered. Longitudinal joints, if used, should be located out of traveled lanes or beneath median barriers to eliminate potentially hazardous vehicle control problems.

   c. The *Standard Specifications* do not permit falsework to be supported from the existing structure unless the Plans and Specifications state otherwise. This requirement eliminates the transmission of vibration from the existing structure to the widening during construction. The existing structure may still be in service.

   d. For narrow widenings where the Plans and Specifications require that the falsework be supported from the original structure (e.g., there are no additional girders, columns, crossbeams, or closure strips), there should be no external rigid supports such as posts or falsework from the ground. Supports from the ground do not permit the widening to deflect with the existing structure when traffic is on the existing structure. This causes the uncured concrete of the widening to crack where it joins the existing structure. Differential dead load deflection during construction should be given consideration.

e. Precast members may be used to widen existing cast-in-place structures. This method is useful when the horizontal or vertical clearances during construction are insufficient to build cast-in-place members.

f. The alignment for diaphragms for the widening shall generally coincide with the existing diaphragms.

g. When using battered piles, estimate the pile tip elevations and ensure that they will have ample clearance from all existing piles, utilities, or other obstructions. Also check that there is sufficient clearance between the existing structure and the pile driving equipment.

B. Seismic Design Criteria for Bridge Widenings

1. Adequacy of Existing Structure

Early in the project, determine whether earthquake loading poses any problems for the structural adequacy of the existing structure (e.g., original unwidened structure). The amount of reinforcement and structural detailing of older structures may not meet the current AASHTO seismic design requirements. It is important that these deficiencies be determined as soon as possible so that remedial/retrofitting measures can be evaluated. It should be noted that for some structures, because of deterioration and/or inadequate details, the widening may not be structurally or economically feasible. In this case, the Bridge Design Engineer should be consulted for possible structure replacement instead of proceeding with widening the structure.

2. Superstructure Widening Without Adding Substructure

No seismic analysis is necessary for this condition. Check the support shelf length required at all piers. Check the need for longitudinal earthquake restrainers and transverse earthquake stops.

3. Superstructure Widening by Adding Column(s) and Substructure

Use the AASHTO/BDM seismic design criteria with appropriate R factors to design and detail the new columns and footings for the maximum required capacity.

Analyze the widening and the existing structure as a combined unit.

If the existing structure is supported by single column piers, and is located in SPC or C (LRFD Seismic Zone 2, 3, or 4), the existing columns <u>should</u> be retrofitted if the existing column does not have adequate ductility to meet current standards.

If the existing structure is supported by multiple column piers, determine the need to retrofit the existing columns as part of the widening as follows:

a. For existing bridges in SPC B or C (LRFD, Zone 2, 3, or 4) that are widened with additional columns and substructure, existing columns <u>should be considered</u> for retrofitting unless calculations or column details indicate that the existing columns have adequate ductility. Nonductile existing columns will likely not be able to carry vertical load if they experience the inelastic deflection that a new (ductile) column can tolerate.

b. Only the columns should be retrofitted. Retrofitting the foundations supporting existing columns is generally too expensive to consider for a widening project. Experience in past earthquakes in California has shown that bridges with columns (only) retrofitted have performed quite well.

c. Approval for retrofitting existing multiple column piers is subject to available funding and approval of the Bridge Design Engineer.

4. Other Criteria

a. If recommended in the foundation report, the superstructure widening with new substructure shall also be checked for differential settlement between the existing structure and the new widened structure. All elements of the structure shall be analyzed and detailed to account for this differential settlement especially on spread footing foundations.

b. Check support width requirements; if there is a need for earthquake restrainers on the existing structure as well as the widened portion, they shall be included in the widening design.

c. The current AASHTO seismic design criteria may result in columns with more reinforcement and larger footings for the widened portion than those on the existing structure. If it is not possible to use larger footings because of limited space, an alternate design concept such as drilled shafts may be necessary.

d. When modifications are made near or on the existing bridge, be careful to isolate any added potential stiffening elements (such as traffic barrier against colmuns).

e. The relative stiffness of the new columns compared to the existing columns should be considered in the combined analysis. The typical column retrofit is steel jacketing with grouted annular space (between the existing column and the steel jacket).

f. When strutted columns (horizontal strut between existing columsn) are encountered, remove the strut and analyze the existing columns for the new unbraced length and retrofit, if necessary. Refer to WSDOT Research Report on Strutted Columns (nearing completion).

C. Substructure

1. Selection of Foundation

a. The type of foundation to be used to support the widening should generally be the same as that of the existing structure unless otherwise recommended by the Geotechnical Engineer. The effects of possible differential settlement between the new and the existing foundations shall be considered.

b. Consider present bridge site conditions when determining new foundation locations. The conditions include: overhead clearance for pile driving equipment, horizontal clearance requirements, working room, pile batters, channel changes, utility locations, existing embankments, and other similar conditions.

2. Scour and Drift

Added piles and columns for widenings at water crossings may alter stream flow characteristics at the bridge site. This may result in pier scouring to a greater depth than experienced with the existing configuration. Added substructure elements may also increase the possibility of trapping drift. The Hydraulics Engineer should be consulted concerning potential problems related to scour and drift on all widenings at water crossings.

D. Superstructure

1. Camber

Accurate prediction of dead load deflection is more important for widenings than for new bridges, since it is essential that the deck grades match [15].

To obtain a smooth transition in transverse direction of the bridge deck, the camber of the girder adjacent to the existing structure shall be adjusted for the difference in camber between new and existing structure. A linear interpolation may be used to adjust the camber of the girders located away from the existing structure. The multipliers for estimating camber of new structure may be taken as shown in table 5.3.5-1.

When large cambers are expected, see Section 5.5.2.A9b.

2. Closure Strip

Except for narrow deck slab widenings (see Section 5.5.2.A9c) a closure strip is required for all cast-in-place widenings. The width shall be the minimum required to accommodate the necessary reinforcement and for form removal. Reinforcement, which extends through the closure strip shall be investigated in accordance with Section 5.5.4A7. Shear shall be transferred across the closure strip by shear friction and/or shear keys.

All falsework supporting the widening shall be released and formwork supporting the closure strip shall be supported from the existing and newly widened structures prior to placing concrete in the closure strip. Because of deck slab cracking experienced in widened concrete decks, closure strips are required unless the mid-span dead load camber is $^{1}/_{2}$ inch or less.

3. Stress Levels and Deflections in Existing Structures

Caution is necessary in determining the cumulative stress levels, deflections, and the need for shoring in existing structural members during rehabilitation projects.

For example, a T-beam bridge was originally constructed on falsework and the falsework was released after the slab concrete gained strength. As part of a major rehabilitation project, the bridge was closed to traffic and the entire slab was removed and replaced without shoring. Without the slab, the stems behave as rectangular sections with a reduced depth and width. The existing stem reinforcement was not originally designed to support the weight of the slab without shoring. After the new slab was placed, wide cracks, eminating from the bottom of the stem opened, indicating that the reinforcement was overstressed. This overstress resulted in a lower load rating for the newly rehabilitated bridge. This example shows the need to shore up the remaining T-beam stems prior to placing the new slab so that excessive deflections do not occur and overstress in the existing reinforcing steel is prevented.

It is necessary to understand how the original structure was constructed, how the rehabilitated structure is to be constructed, and the cumulative stress levels and deflections in the structure from the time of original construction through rehabilitation.

E. Stability of Widening

For relatively narrow box girder and T-beam widenings, symmetry about the vertical axis should be maintained because lateral loads are critical during construction. When symmetry is not possible, use pile cap connections, lateral connections, or special falsework. A minimum of two webs is generally recommended for box girder widenings. For T-beam widenings that require only one additional web, the web should be centered at the axis of symmetry of the slab. Often the width of the closure strip can be adjusted to accomplish this.

### 5.5.3 Removing Portions of the Existing Structure

Portions of the existing structure to be removed shall be clearly indicated on the plans. Where a clean break line is required, a 3/4″ deep saw cut shall be specified for a slab with normal wear and a 1/2″ deep saw cut for worn roadway slabs. In no case, however, shall the saw blade cut or nick the main transverse top slab reinforcement. The special provisions shall state that care will be taken not to damage any reinforcement which is to be saved. Hydromilling is preferred where reinforcing bar cover is shallow and can effectively remove delaminated decks because of the good depth control it offers. When greater depths of slab are to be removed, special consideration should be given to securing exposed reinforcing bars to prevent undue vibration and subsequent fatigue cracks from occurring in the reinforcing bars.

The current General Special Provisions should be reviewed for other specific requirements on slab removal.

Removal of any portion of the main structural members should be held to a minimum. Careful consideration shall be given to the construction conditions, particularly when the removal affects the existing frame system. In extreme situations, preloading by jacking is acceptable to control stresses and deflections during the various stages of removal and construction. Removal of the main longitudinal slab reinforcement should be kept to a minimum. See "Slab Removal Detail," Figure 5.5-1, for the limiting case for the maximum allowable removal.

The plans should include a note that critical dimensions and elevations are to be verified in the field prior to the fabrication of precast units or expansion joint assemblies.

In cases where an existing sidewalk is to be removed but the supporting slab under the sidewalk is to be retained, district personnel should check the feasibility of removing the sidewalk. Prior to design, Region personnel should make recommendations on acceptable removal methods and required construction equipment. The plans and specifications should then be prepared to accommodate these recommendations. This will ensure the constructibility of plan details and the adequacy of the specifications.

### 5.5.4 Attachment of Widening to Existing Structure

A. General

1. Lap and Mechanical Splices

To attach a widening to an existing structure, the first choice is to utilize existing reinforcing bars by splicing new bars to existing. Lap splices or mechanical splices should be used. However, it may not always be possible to splice to existing reinforcing bars and spacing limitations may make it difficult to use mechanical splices.

2. Welding Reinforcement

   Existing reinforcing steel may not be readily weldable. Mechanical splices should be used wherever possible. If welding is the only feasible means, the chemistry of the reinforcing steel must be analyzed and acceptable welding procedures developed.

3. Drilling Into Existing Structure

   It may be necessary to drill holes and set dowels in epoxy resin in order to attach the widening to the existing structure.

   When drilling into heavily reinforced areas, chipping should be specified to expose the main reinforcing bars. If it is necessary to drill through reinforcing bars or if the holes are within 4 inches of an existing concrete edge, core drilling should be specified. Core drilled holes shall be roughened before resin is applied. If this is not done, a dried residue, which acts as a bond breaker and reduces the load capacity of the dowel, will remain. Generally, the drilled holes are $^1/_8$ inch in diameter larger than the dowel diameter for #5 and smaller dowels and $^1/_4$ inch in diameter larger than the dowel diameter for #6 and larger dowels.

   In special applications requiring drilled holes greater than $1^1/_2''$ inch diameter or deeper than 2 feet, core drilling shall be specified. These holes should also be intentionally roughened prior to applying epoxy resin.

   Core drilled holes should have a minimum clearance of 3 inches from the edge of the concrete and 1-inch clearance from existing reinforcing bars in the existing structure. These clearances should be noted in the plans.

4. Dowelling Reinforcing Bars Into the Existing Structure

   a. Dowel bars shall be set with an approved epoxy resin. The existing structural element shall be checked for its adequacy to transmit the load transferred to it from the dowel bars.

   b. Dowel spacing and edge distance affect the allowable tensile dowel loads [14]. Allowable tensile loads, dowel bar embedments, and drilled hole sizes for reinforcing bars (Grade 60) used as dowels and set with an approved epoxy resin are shown in Table 5.5-1. These values are based on an edge clearance greater than 3 inch, a dowel spacing greater than 6 inch, and are shown for both uncoated and epoxy coated dowels. Table 5.5-2 lists dowel embedment lengths when the dowel spacing is less than 6 inch. Note that in Table 5.5-2 the edge clearance is equal to or greater than 3 inch, because this is the minimum edge clearance for a drilled hole from a concrete edge.

      If it is not possible to obtain these embedments, such as for traffic railing dowels into existing deck slabs, the allowable load on the dowel shall be reduced by the ratio of the actual embedment divided by the required embedment.

c. The embedments shown in Table 5.5-1 and -2 are based on dowels embedded in concrete with $f_c'$=4,000 psi.

Allowable Tensile Load for Dowels Set With Epoxy Resin $f_c'$=4,000 psi, Gr 60 Reinforcing Bars, Edge Clearance ≥ 3 in., and Spacing ≥ 6 in.[14]
Table 5.5-1

| Bar Size | Allowable Design Tensile Load, T* (kips) | Drill Hole Size (in) | Required Embedment, Le** Uncoated (in) | Epoxy Coated (in) |
|---|---|---|---|---|
| 4 | 12.0 | $^5/_8$ | 7 | 8 |
| 5 | 18.6 | $^3/_4$ | 8 | 9 |
| 6 | 26.4 | 1 | 9 | 10 |
| 7 | 36.0 | $1^1/_8$ | 11 | 12 |
| 8 | 47.4 | $1^1/_4$ | 13 | $14^1/_2$ |
| 9 | 60.0 | $1^3/_8$ | 16 | $17^1/_2$ |

*Allowable Tensile Load (Strength Design) = $(f_y)(A_s)$.
**Based on removed cover. In cases where concrete cover is not removed, the designer should add the cover thickness to the required embedment.

Allowable Tensile Load for Dowels Set With Epoxy Resin $f_c'$=4,000 psi, Gr 60 Reinforcing Bars, Edge Clearance ≥ 3 in., and Spacing < 6 in.[14]
Table 5.5-2

| Bar Size | Allowable Design Tensile Load, T* (kips) | Drill Hole Size (in) | Required Embedment, Le** Uncoated (in) | Epoxy Coated (in) |
|---|---|---|---|---|
| 4 | 12.0 | $^5/_8$ | $9^1/_2$ | $10^1/_2$ |
| 5 | 18.6 | $^3/_4$ | $10^1/_2$ | $11^1/_2$ |
| 6 | 26.4 | 1 | $11^1/_2$ | $12^1/_2$ |
| 7 | 36.0 | $1^1/_8$ | $13^1/_2$ | 15 |
| 8 | 47.4 | $1^1/_4$ | $16^1/_2$ | 18 |
| 9 | 60.0 | $1^3/_8$ | 20 | 22 |

*Allowable Tensile Load (Strength Design) = $(f_y)(A_s)$.
**Based on removed cover. In cases where concrete cover is not removed, the designer should add the cover thickness to the required embedment.

5. Shear Transfer Across a Dowelled Joint

Shear should be carried across the joint by shear friction on an intentionally roughened surface instead of depending on the dowels to transmit the shear force. Chipping shear keys in the existing concrete can also be used to transfer shear across a dowelled joint, but is expensive.

6. Preparation of Existing Surfaces for Concreting

   See "Removing Portions of Existing Structure" in the General Special Provisions for requirements. Unsound, damaged, dirty, porous, or otherwise undesirable old concrete should be removed, and the remaining concrete surface should be clean, free of laitance, and intentionally roughened to ensure proper bond between the old and new concrete surfaces.

7. Control of Shrinkage and Deflection on Connecting Reinforcement

   Dowels that are fixed in the existing structure may be subject to shear as a result of longitudinal shrinkage and vertical deflection when the falsework is removed. These shear forces may result in a reduced tensile capacity of the connection. When connecting the transverse reinforcing bars across the closure strip is unavoidable, the interaction between shear and tension in the dowel or reinforcing bar should be checked. The use of wire rope or sleeved reinforcement may be acceptable, subject to approval by your supervisor.

   Where possible, transverse reinforcing bars should be spliced to the existing reinforcing bars in a blocked-out area which can be included in the closure strip. Nominal, shear friction, temperature and shrinkage, and distribution reinforcing bars should be bent into the closure strip.

   Rock bolts may be used to transfer connection loads deep into the existing structure, subject to the approval of your supervisor.

8. Post-Tensioning

   Post-tensioning of existing crossbeams may be utilized to increase the moment capacity and to eliminate the need for additional substructure. Generally, an existing crossbeam can be core drilled for post-tensioning if it is less than 30 feet long. The amount of drift in the holes alignment may be approximately 1 inch in 20 feet. For crossbeams longer than 30 feet, external post-tensioning should be considered.

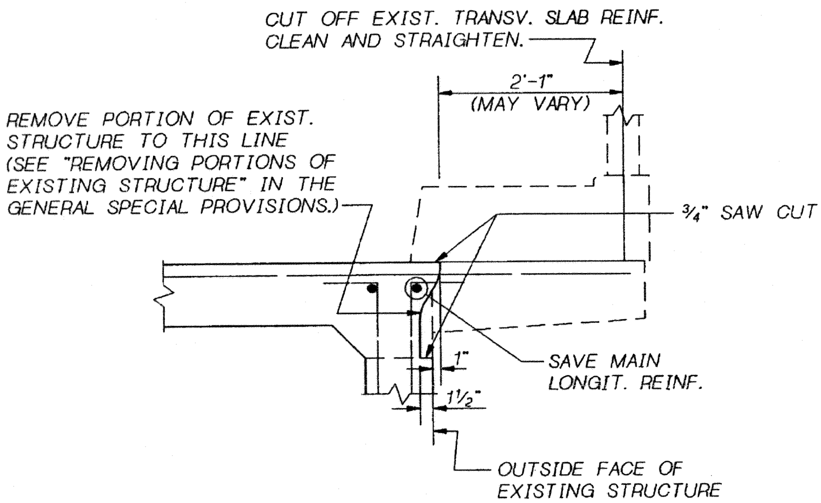
   For an example of this application, refer to Contract 3846, Bellevue Transit Access — Stage 1.

B. Connection Details

The details on the following sheets are samples of details which have been used for widening bridges. They are informational and are not intended to restrict the designer's judgment.

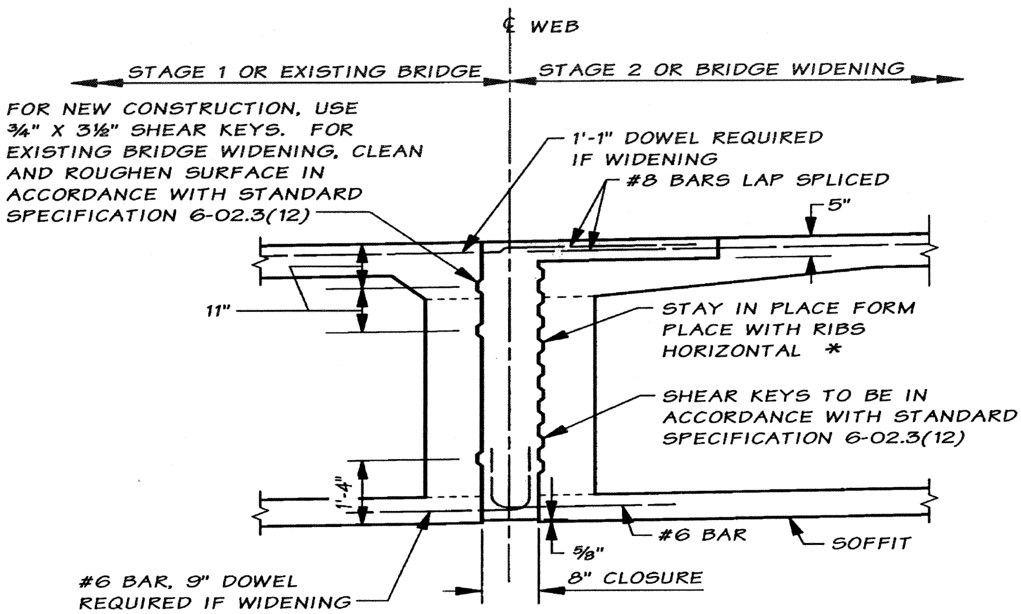
Slab Removal Detail
Figure 5.5.-1

1. Box Girder Bridges




Figures 5.5-2, -3, -4, and -5 show typical details for widening box girder bridges.

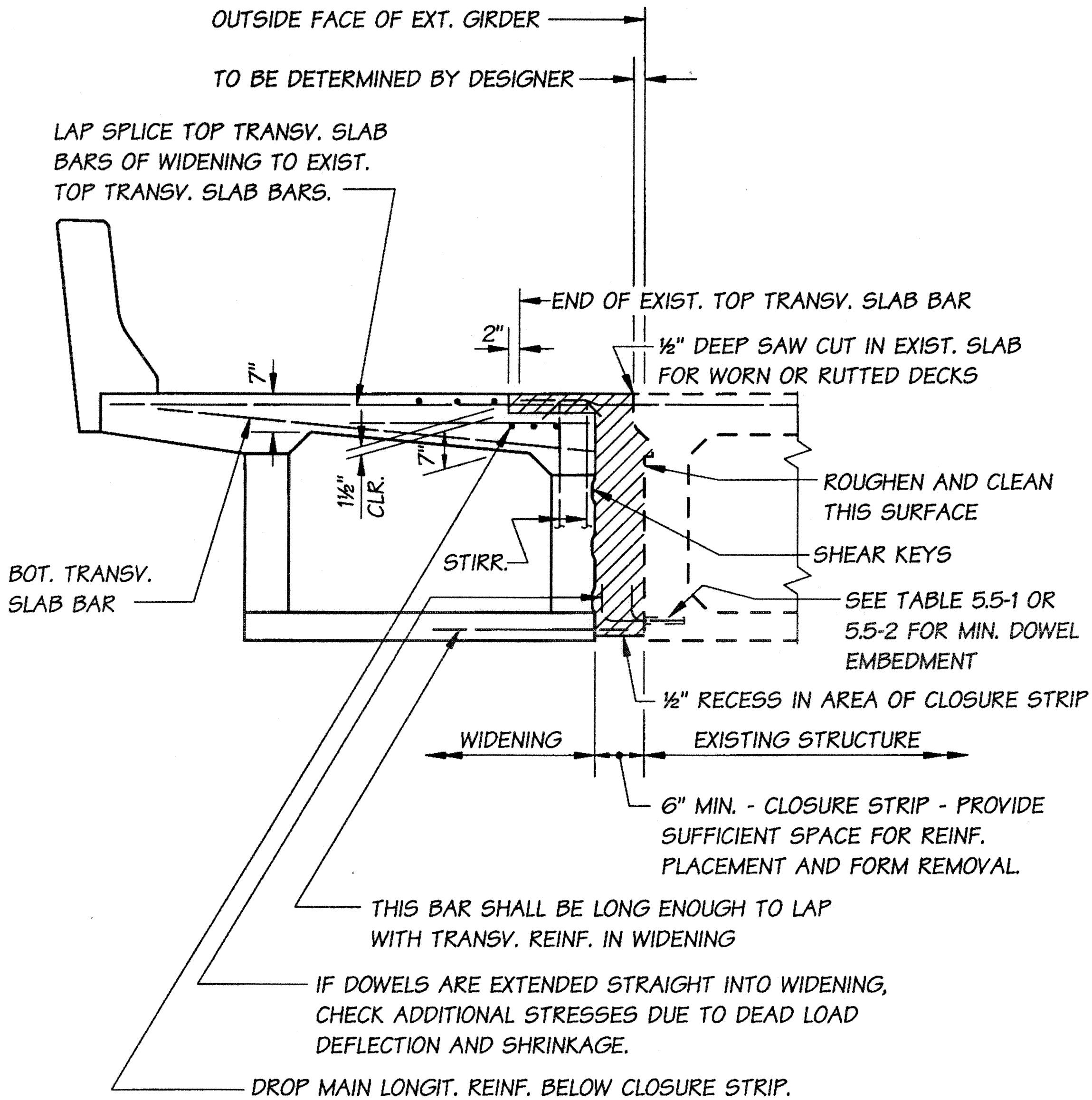
Box Girder Section in Span
Figure 5.5-2



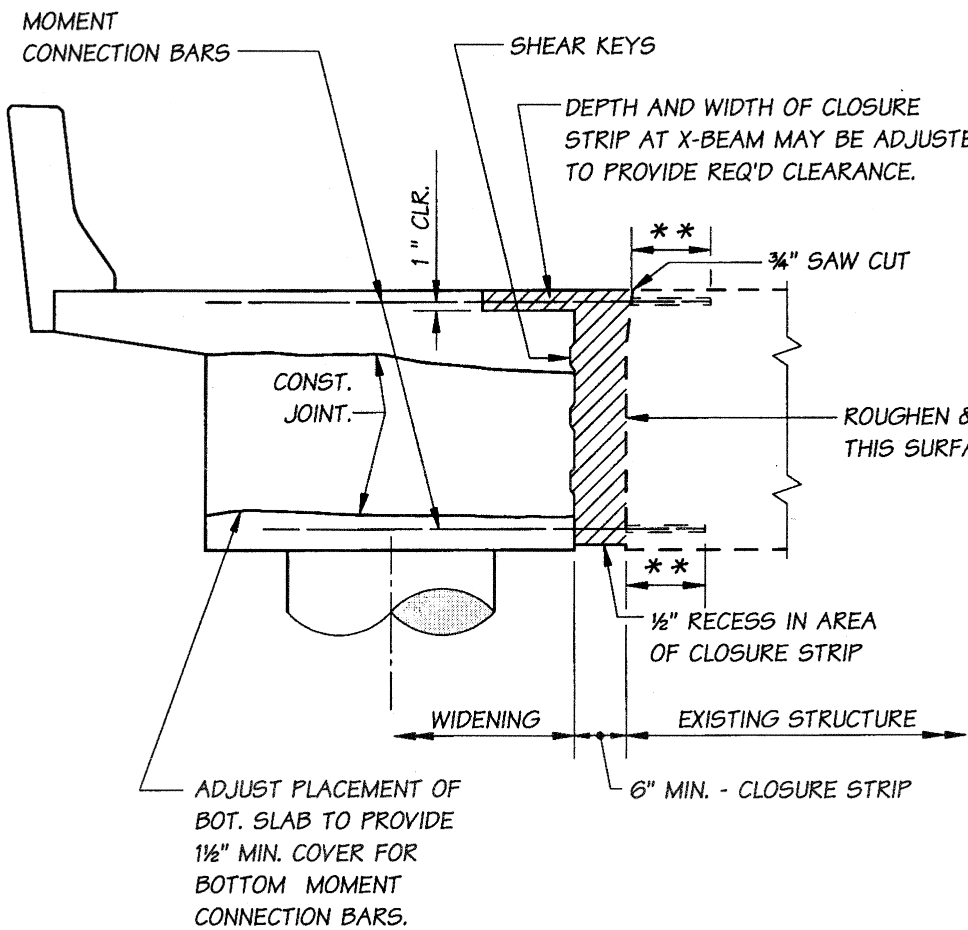

STAY IN PLACE FORM DETAIL
FOR BOX GIRDER STAGED CONSTRUCTION
OR WIDENING

* STAY IN PLACE FORMS SHALL BE SOLID GALVANIZED SHEET METAL. FORMS MUST BE VERTICALLY BRACED AS NECCESSARY TO PREVENT BOWING DURING CONCRETE PLACEMENT. TIMBER BRACING MUST BE REMOVED. IF STEEL WALES OR TIES ARE USED, THEY MAY BE LEFT IN PLACE. THE CONTRACTOR SHALL SUBMIT DESIGN CALCULATIONS IN ACCORDANCE WITH STANDARD SPECIFICATIONS 6-02.3(16) AND 6-02.3(17).

Box Girder Section Through X-Beam
See Box Girder Section in Span for additional details.
Figure 5.5-3




Welding or mechanical butt splice are preferred over dowelling for the main reinforcement in crossbeams and columns when it can be done in the horizontal or flat position. It shall be allowed only when the bars to

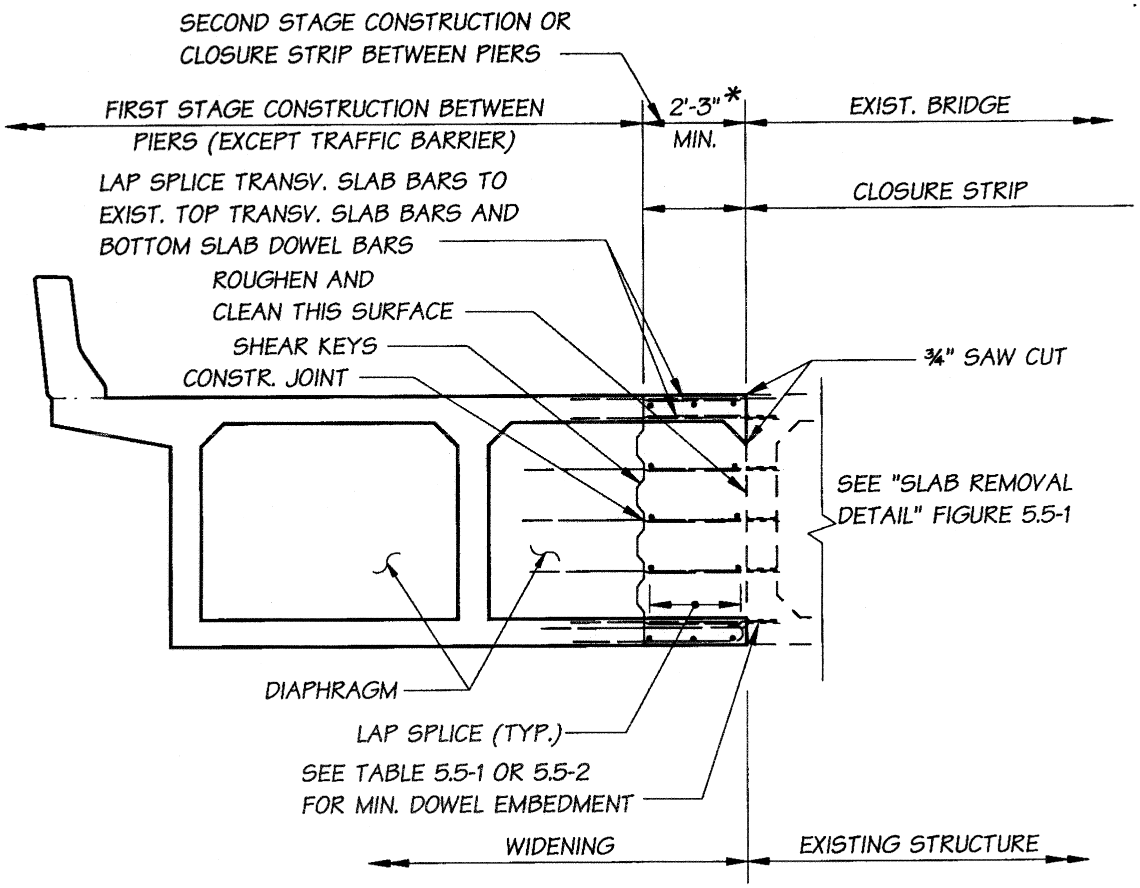



be welded are free from restraint at one end during the welding process.

**If bars are to be dowelled, provide a sufficient embedment depth for moment connection bars into existing structure that will provide the required moment capacity in the existing structure. See Table 5.5-1 or 5.5-2.

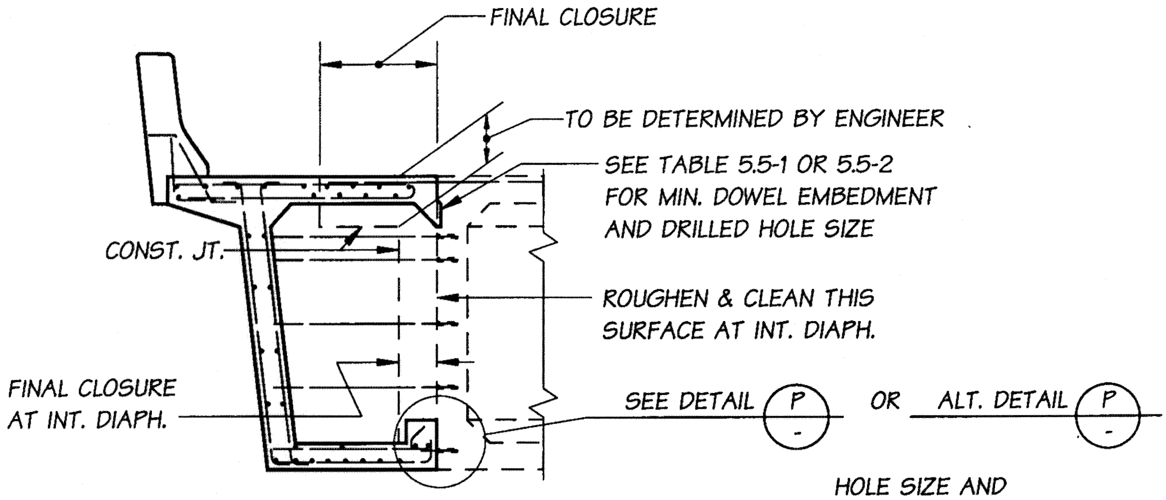
Box Girder Section in Span at Diaphragm Alternate I
Figure 5.5-4
Box Girder Section in Span at Diaphragm Alternate II
Figure 5.5-5

2. Flat Slab Bridges

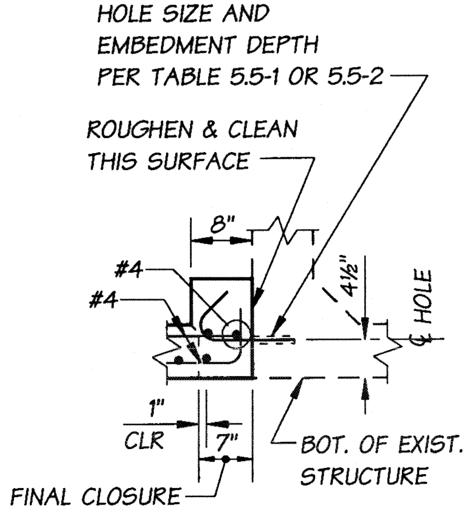



* IF LAP SPLICE EXCEEDS 2'-0", INCREASE WIDTH OF CLOSURE STRIP TO ACCOMMODATE INCREASED LAP SPLICE.

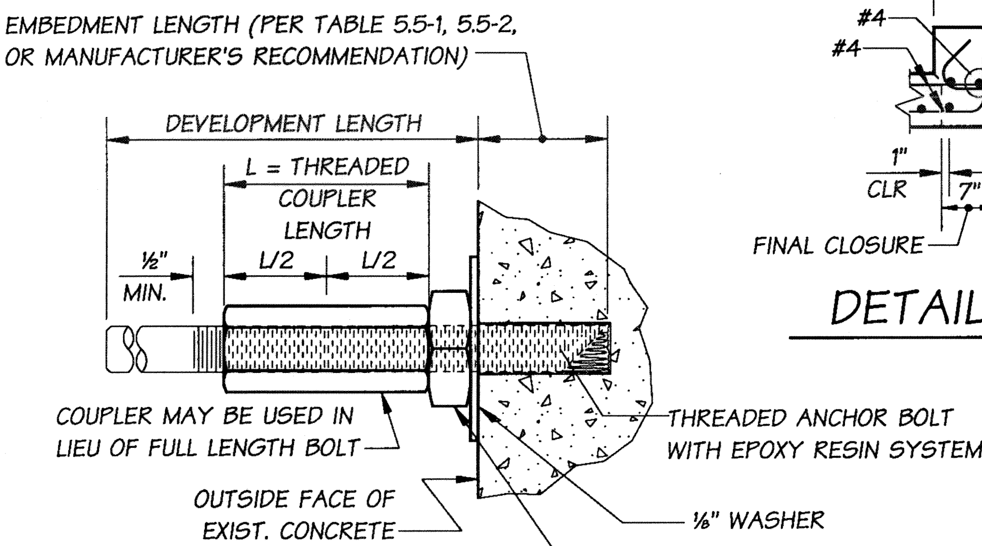
It is not necessary to remove any portion of the existing slab to expose the existing transverse reinforcing bars for splicing purposes, because the transverse slab reinforcement is only distribu-




NOTE: THIS ALTERNATE APPLIES TO NARROW WIDENINGS WHERE SHEAR IN THE EXTERIOR WEB IS NOT CRITICAL. THIS IS TYPICAL FOR SHORT TO MEDIUM SPANS OR WHERE THE EXISTING SLAB OVERHANG IS CONSIDERABLY LESS THAN HALF THE WEB SPACING.




DETAIL P -




ALT. DETAIL P -

NOTE: INSTALL ANCHOR BOLT WITH EPOXY RESIN SYSTEM PER MANUFACTURER'S RECOMMENDATIONS IN DRY CONDITIONS.

tion reinforcement. The transverse slab reinforcement for the widening may be dowelled directly into the existing structure without meeting the normal splice requirements.

For the moment connection details, see Figure 5.5-6 for "Flat Slab — Section through X-Beam."

Note: Falsework shall be maintained under pier crossbeams until closure pour is made and cured 10 days.

Flat Slab — Section through X-Beam

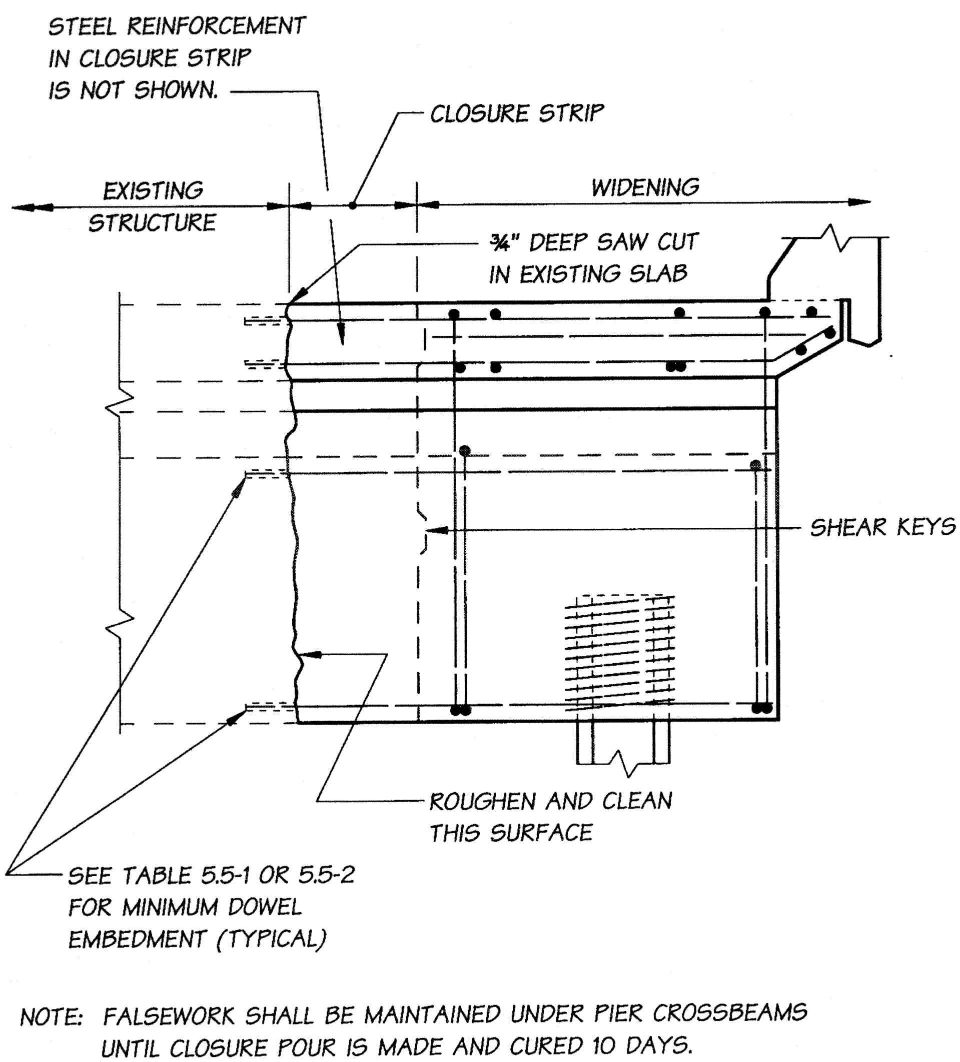


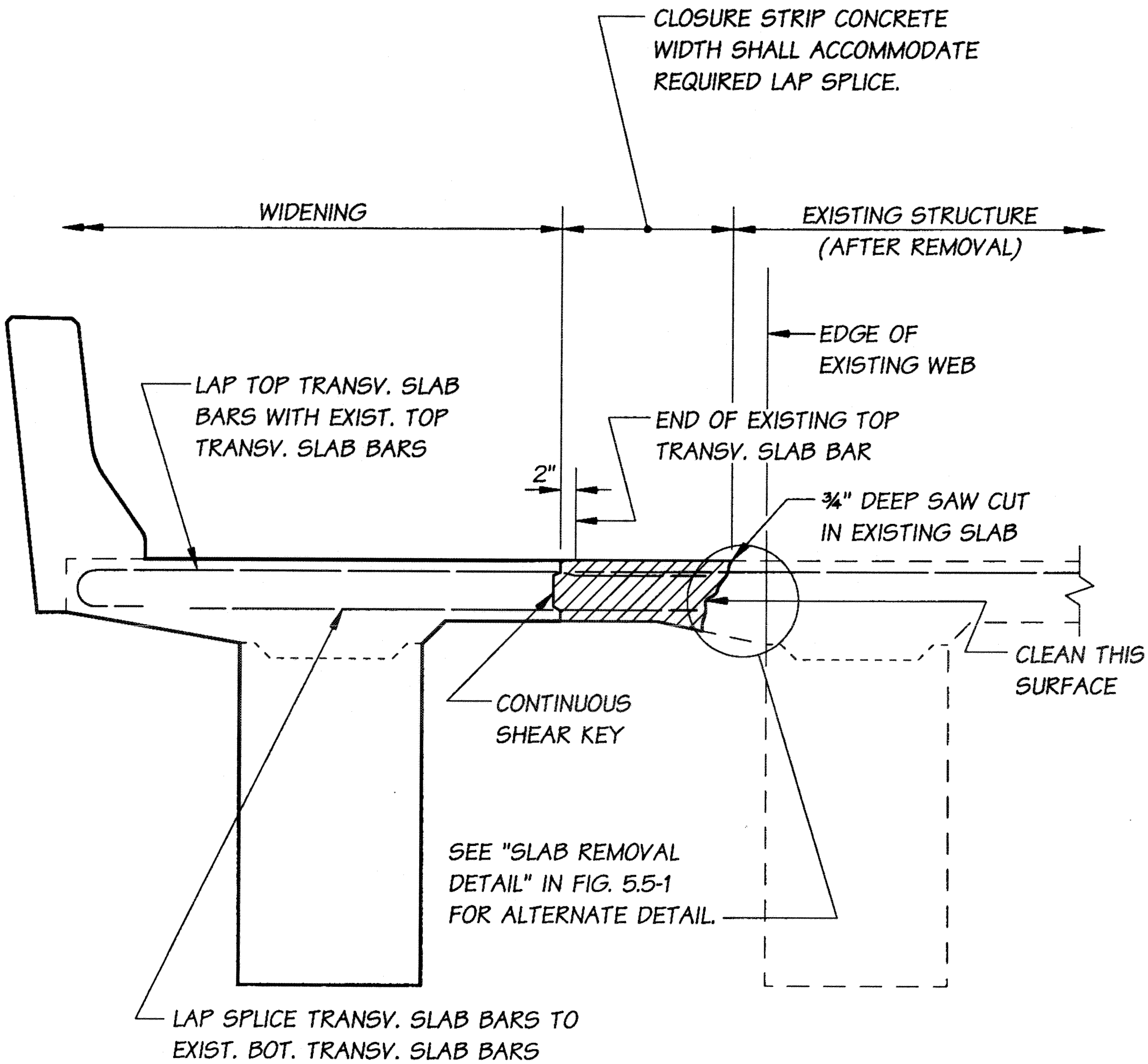

Figure 5.5-6

3. T-Beam Bridges

   Use details similar to those for box girder bridges for crossbeam connections. See Figure 5.5-7 for slab connection detail.

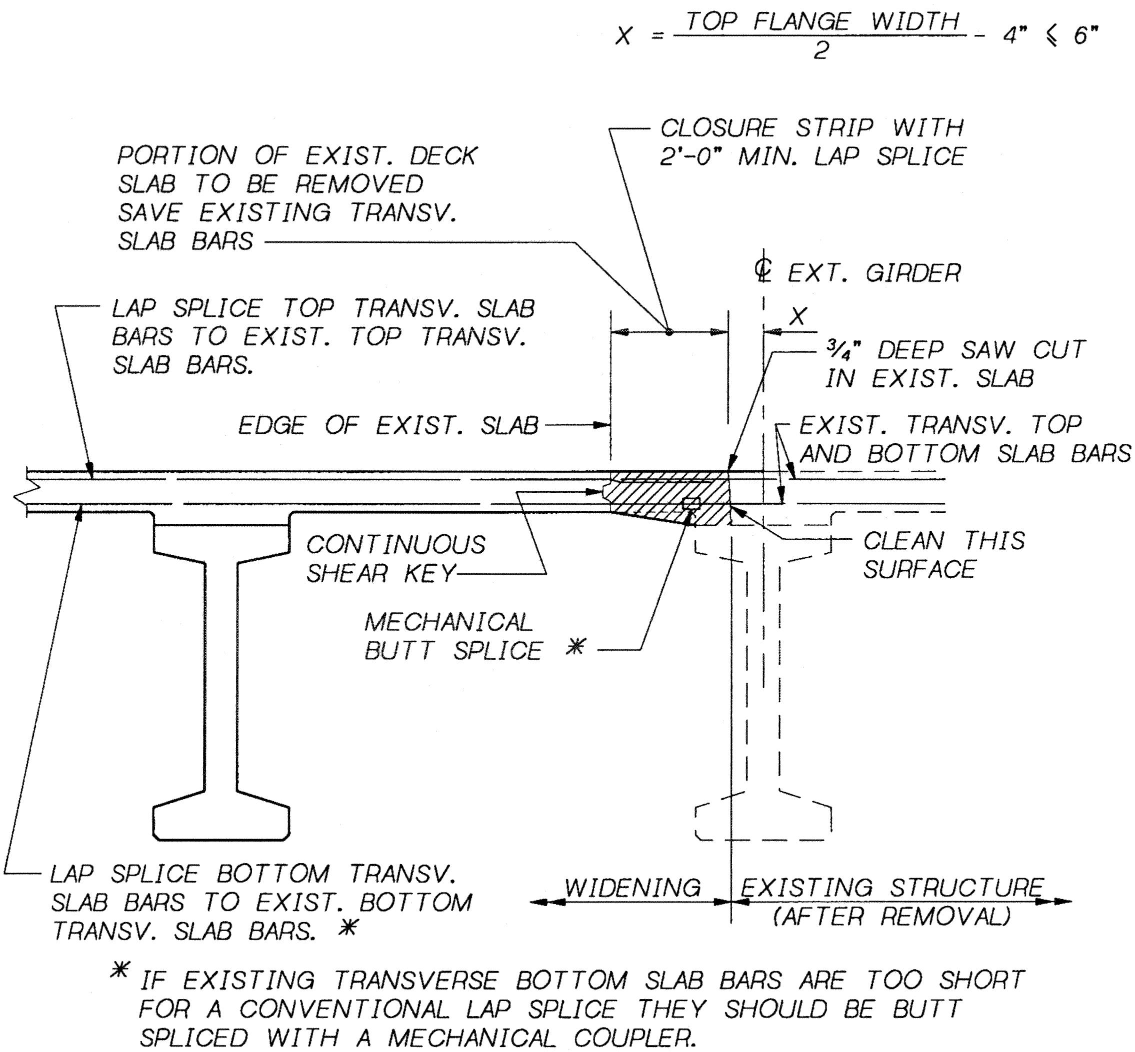
T-Beam — Section in Span
Figure 5.5-7

4. Prestress Concrete Girder Bridges




Use details similar to those for box girder bridges for crossbeam moment connections and use details similar to those in Figure 5.5-8 for connecting to the slab.

Prestressed Girder — Section in Span
Figure 5.5-8



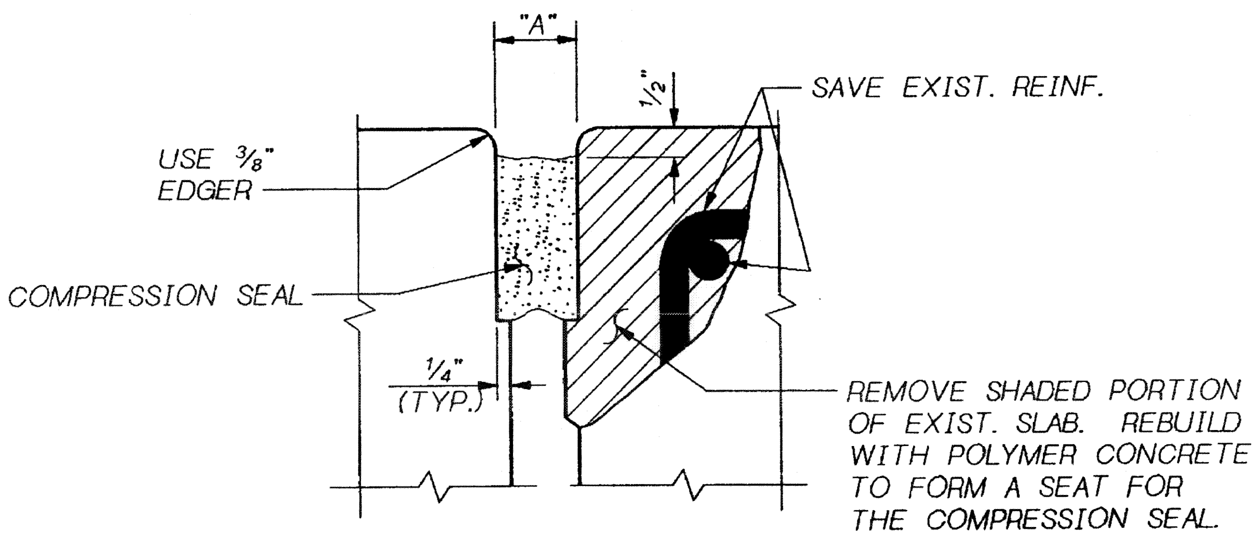

## 5.5.5 Expansion Joints

The designer should determine if existing expansion joints can be eliminated. It will be necessary to determine what modifications to the structure are required to provide an adequate functional system when existing joints are eliminated.

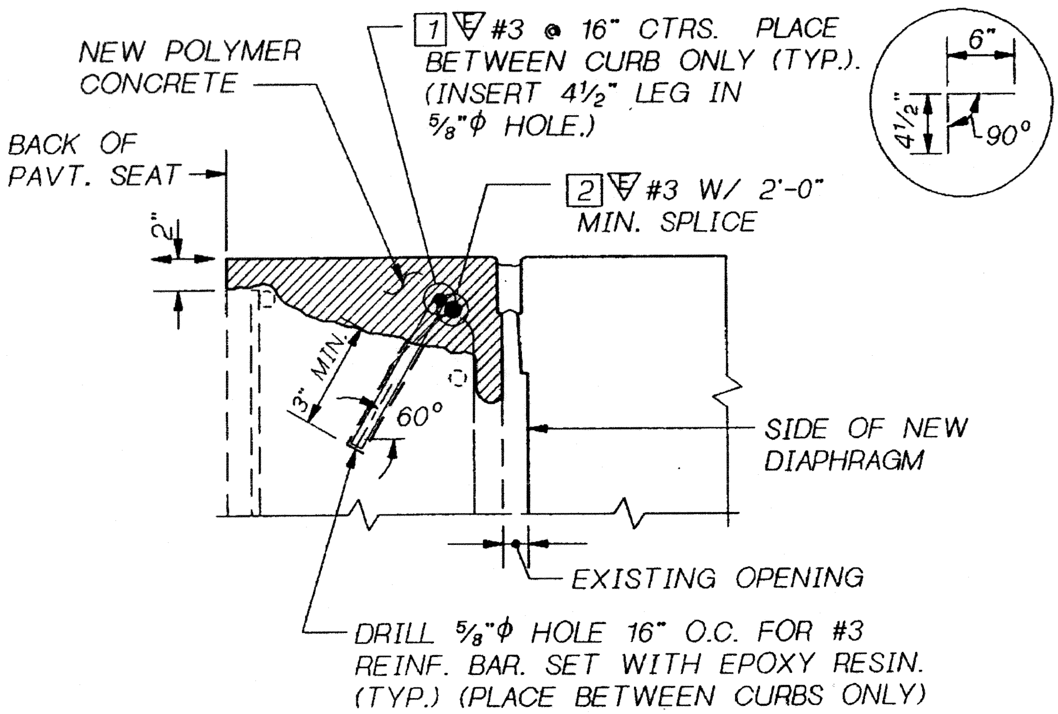
For expansion joint design, see Section 8.4.1 "Expansion Joints." Very often on widening projects it is necessary to chip out the existing concrete deck and rebuild the joint. Figures 5.5-9 and 5.5-10 show details for rebuilding joint openings for compression seal expansion joints.

If a widening project includes an overlay, the expansion joint may have to be raised, modified or replaced. See the Joint Specialist for plan details that are currently being used to modify or retrofit existing expansion joints.

Expansion Joint
Detail Shown for Compression Seal — Existing Reinforcing Steel Saved
Figure 5.5-9
Expansion Joint




Detail shown for compression seal with new reinforcing steed added.
Figure 5.5-10




### 5.5.6 Possible Future Widening for Current Designs

For current projects that include sidewalks (and where it is anticipated that the structure may be modified or widened in the future), provide a smooth rather than a rough construction joint between the sidewalk and the slab. This will normally pertain to flat slab bridges or where the sidewalk width exceeds the slab cantilever overhang.

### 5.5.7 Bridge Widening Falsework

For widenings which do not have additional girders, columns, crossbeams, or closure pours, flasework should be supported by the existing bridge. There should be an external support from the ground. The reason is that the ground support will not allow the widening to deflect the existing bridge when traffic is on the bridge. This will cause the "green" concrete to crack where it joins the existing bridge. Designer should contact the bridge construction support unit regarding fasework associated with widenings.

## 5.5.8 Existing Bridge Widenings

The following listed bridge widenings are included as aid to the designer. These should not be construed as the only acceptable methods of widening; there is no substitute for the designer's creativity or ingenuity in solving the challenges posed by bridge widenings.

| Bridge | SR | Contract No. | Type of Bridge | Unusual Features |
|---|---|---|---|---|
| NE 8th Street U'Xing | 405 | 9267 | Ps. Gir. | Pier replacements |
| Higgins Slough | 536 | 9353 | Flat Slab | |
| ER17 and AR17 O-Xing | 5 | 9478 | Box Girder | Middle and outside widening. |
| SR 538 O-Xing | 5 | 9548 | T-Beam | Unbalanced widening section support at diaphragms until completion of closure pour. |
| B-N O'Xing | 5 | 9566 | Box Girder | Widened with P.S. Girders, X-beams, and diaphragms not in line with existing jacking required to manipulate stresses, added enclusure walls. |
| Blakeslee Jct. E/W | 5 | 9638 | T-Beam and Box Girder | Post-tensioned X-beam, single web. |
| B-N O'Xing | 18 | 9688 | Box Girder | |
| SR 536 | | 9696 | T-Beam | Similar to Contract 9548. |
| LE Line over Yakima River | 90 | 9806 | Box Girder | Pier shaft. |
| SR 18 O-Xing | 90 | 9823 | P.S. Girder | Lightweight concrete. |
| Hamilton Road O-Xing | 5 | 9894 | T-Beam | Precast girder in one span. |
| Dillenbauch Creek | 5 | | Flat Slab | |
| Longview Wye SR 432 U-Xing | 5 | | P.S. Girder | Bridge lengthening. |
| Klickitat River Bridge | 142 | | P.S. Girder | Bridge replacement. |
| Skagit River Bridge | 5 | | Steel Truss | Rail modification. |
| B-N O-Xing at Chehalis | 5 | | | Replacement of thru steel girder span with stringer span. |
| Bellevue Access EBCD Widening and Pier 16 Modification | 90 | 3846 | Flat Slab and Box Girder | Deep, soft soil. Stradle best replacing single column. |
| Totem Lake/NE 124th I/C | 405 | 3716 | T-Beam | Skew = 55 degrees. |
| Pacific Avenue I/C | 5 | 3087 | Box Girder | Complex parallel skewed structures. |
| SR 705/SR 5 SB Added Lane | 5 | 3345 | Box Girder | Multiple widen structures. |
| Mercer Slough Bridge 90/43S | | 3846 | CIP Conc. Flat Slab | Tapered widening of flat slab outrigger pier, combined footings. |
| Spring Street O-Xing No. 5/545SCD | | 3845 | CIP Conc. Box Girder | Tapered widening of box girder with hingers, shafts. |
| Fishtrap Creek Bridge 546/8 | | 3661 | P.C. Units | Widening of existing P.C. Units. Tight constraints on substructure. |
| Columbia Drive O-Xing 395/16 | | 3379 | Steel Girder | Widening/Deck replacement using standard rolled sections. |

| Bridge | SR | Contract No. | Type of Bridge | Unusual Features |
|---|---|---|---|---|
| S 74th-72nd St. O-Xing No. 5/426 | | 3207 | CIP Haunched Con. Box Girder | Haunched P.C. P.T. Bath Tub girder sections. |
| Pacific Avenue O-Xing No. 5/332 | | 3087 | CIP Conc. Box Girder | Longitudinal joint between new and existing. |
| Tye River Bridges 2/126 and 2/127 | | 3565 | CIP Conc. Tee Beam | Stage construction with crown shift. |
| SR 20 and BNRR O-Xing No. 5/714 | | 9220 | CIP Conc. Tee Beam | Widened with prestressed girders raised crossbeam. |
| NE 8th St. U'Xing No. 405/43 | | 9267 | Prestressed Girders | Pier replacement — widening. |
| So. 212th St. U'Xing SR 167 | | 3967 | Prestressed Girders | Widening constructed as stand alone structure. Widening column designed as strong column for retrofit. |
| SE 232nd St. SR 18 | | 5801 | CIP Conc. Post-tensioned Box | Skew = 50 degree. Longitudinal "link pin" deck joint between new and existing to accommodate new creep. |
| Obdashian Bridge 2/275 | | N/A 1999 | CIP Post-tensioned Box | Sidewalk widening with pipe struts. |

P65:DP/BDM5

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*P65:DP/BDM5*

| Size | Weight (lbs/ft) | Nominal Diameter* (in) | Outside Diameter (in) | Area (in²) | Maximum Bar Length (ft) | Normal Bar Length (ft) |
|---|---|---|---|---|---|---|
| #3 | 0.376 | 3/8″ | 0.42 | 0.11 | 40′ | 30′ |
| #4 | 0.668 | 1/2″ | 0.56 | 0.20 | 40′ | 40′ |
| #5 | 1.043 | 5/8″ | 0.70 | 0.31 | 60′ | 40′ |
| #6 | 1.502 | 3/4″ | 0.83 | 0.44 | 60′ | 60′ |
| #7 | 2.044 | 7/8″ | 0.96 | 0.60 | 60′ | 60′ |
| #8 | 2.670 | 1″ | 1.10 | 0.79 | 72′** | 60′ |
| #9 | 3.400 | 1.13 (1 1/8″) | 1.24 | 1.00 | 72′** | 60′ |
| #10 | 4.303 | 1.27 (1 1/4″) | 1.40 | 1.27 | 72′** | 60′ |
| #11 | 5.313 | 1.41 (1 3/8″) | 1.55 | 1.56 | 90′** | 60′ |
| #14 | 7.650 | 1.69 (1 3/4″) | 1.86 | 2.25 | 90′** | 60′ |
| #18 | 13.600 | 2.26 (1 1/4″) | 2.48 | 4.00 | 90′** | 60′ |

*Normally 1/8 per bar size number.

**Requires large special order. Since these lengths may pose problems in transporting and handling, get your supervisor's approval before using them. See Chapter 5, Section 5.1.2F.

***Note:*** For sizes > #9, area and weight are based on the decimal diameter.

Table 5.1-A1

**(Reinforcing Bars AASHTO M31)**

**Bar Size**

| | #3 | #4 | #5 | #6 | #7 | #8 | #9 | #10 | #11 | #14 | #18 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Spacing 3″ | 0.44 | 0.80 | | | | | | | | | |
| 3 1/4 | 0.41 | 0.74 | 1.14 | | | | | | | | |
| 3 1/2 | 0.38 | 0.69 | 1.06 | 1.51 | 2.06 | | | | | | |
| 3 3/4 | 0.35 | 0.64 | 0.99 | 1.41 | 1.92 | 2.53 | 3.20 | | | | |
| 4 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 | | |
| 4 1/4 | 0.31 | 0.56 | 0.88 | 1.24 | 1.69 | 2.23 | 2.82 | 3.59 | 4.40 | | |
| 4 1/2 | 0.29 | 0.53 | 0.83 | 1.17 | 1.60 | 2.11 | 2.67 | 3.39 | 4.16 | 6.00 | |
| 4 3/4 | 0.28 | 0.51 | 0.78 | 1.11 | 1.52 | 2.00 | 2.53 | 3.21 | 3.94 | 5.68 | |
| 5 | 0.26 | 0.48 | 0.74 | 1.06 | 1.44 | 1.90 | 2.40 | 3.05 | 3.74 | 5.40 | |
| 5 1/4 | 0.25 | 0.46 | 0.71 | 1.01 | 1.37 | 1.81 | 2.29 | 2.90 | 3.57 | 5.14 | |
| 5 1/2 | 0.24 | 0.44 | 0.68 | 0.96 | 1.31 | 1.72 | 2.18 | 2.77 | 3.40 | 4.91 | |
| 5 3/4 | 0.23 | 0.42 | 0.65 | 0.92 | 1.25 | 1.65 | 2.09 | 2.65 | 3.26 | 4.70 | 8.35 |
| 6 | 0.22 | 0.40 | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 | 4.50 | 8.00 |
| 6 1/2 | 0.20 | 0.37 | 0.57 | 0.81 | 1.11 | 1.46 | 1.85 | 2.35 | 2.88 | 4.15 | 7.38 |
| 7 | 0.19 | 0.34 | 0.53 | 0.75 | 1.03 | 1.35 | 1.71 | 2.18 | 2.67 | 3.86 | 6.86 |
| 7 1/2 | 0.18 | 0.32 | 0.50 | 0.70 | 0.96 | 1.26 | 1.60 | 2.03 | 2.50 | 3.60 | 6.40 |
| 8 | 0.17 | 0.30 | 0.47 | 0.66 | 0.90 | 1.19 | 1.50 | 1.91 | 2.34 | 3.38 | 6.00 |
| 8 1/2 | 0.16 | 0.28 | 0.44 | 0.62 | 0.85 | 1.12 | 1.41 | 1.79 | 2.20 | 3.18 | 5.65 |
| 9 | 0.15 | 0.27 | 0.41 | 0.59 | 0.80 | 1.05 | 1.33 | 1.69 | 2.08 | 3.00 | 5.33 |
| 9 1/2 | 0.14 | 0.25 | 0.39 | 0.56 | 0.76 | 1.00 | 1.26 | 1.60 | 1.97 | 2.84 | 5.05 |
| 10 | 0.13 | 0.24 | 0.37 | 0.53 | 0.72 | 0.95 | 1.20 | 1.52 | 1.87 | 2.70 | 4.80 |
| 10 1/2 | 0.13 | 0.23 | 0.35 | 0.50 | 0.69 | 0.90 | 1.14 | 1.45 | 1.78 | 2.57 | 4.57 |
| 11 | 0.12 | 0.22 | 0.34 | 0.48 | 0.65 | 0.86 | 1.09 | 1.39 | 1.70 | 2.45 | 4.36 |
| 11 1/2 | 0.11 | 0.21 | 0.32 | 0.46 | 0.63 | 0.82 | 1.04 | 1.33 | 1.63 | 2.35 | 4.17 |

$A_s$ Per Foot of Bar
Table 5.1-A2

| No. \ Size | #3 | #4 | #5 | #6 | #7 | #8 | #9 | #10 | #11 | #14 | #18 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 | 2.25 | 4.00 |
| 2 | 0.22 | 0.40 | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 | 4.50 | 8.00 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 | 6.75 | 12.00 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 | 9.00 | 16.00 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 | 11.25 | 20.00 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 | 13.50 | 24.00 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 | 15.75 | 28.00 |
| 8 | 0.88 | 1.60 | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 | 18.00 | 32.00 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 | 20.25 | 36.00 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 | 22.50 | 40.00 |
| 11 | 1.21 | 2.20 | 3.41 | 4.84 | 6.60 | 8.69 | 11.00 | 13.97 | 17.16 | 24.75 | 44.00 |
| 12 | 1.32 | 2.40 | 3.72 | 5.28 | 7.20 | 9.48 | 12.00 | 15.24 | 18.72 | 27.00 | 48.00 |
| 13 | 1.43 | 2.60 | 4.03 | 5.72 | 7.80 | 10.27 | 13.00 | 16.51 | 20.28 | 29.25 | 52.00 |
| 14 | 1.54 | 2.80 | 4.34 | 6.16 | 8.40 | 11.06 | 14.00 | 17.78 | 21.84 | 31.50 | 56.00 |
| 15 | 1.65 | 3.00 | 4.65 | 6.60 | 9.00 | 11.85 | 15.00 | 19.05 | 23.40 | 33.75 | 60.00 |
| 16 | 1.76 | 3.20 | 4.96 | 7.04 | 9.60 | 12.64 | 16.00 | 20.32 | 24.96 | 36.00 | 64.00 |
| 17 | 1.87 | 3.40 | 5.27 | 7.48 | 10.20 | 13.43 | 17.00 | 21.59 | 26.52 | 38.25 | 68.00 |
| 18 | 1.98 | 3.60 | 5.58 | 7.92 | 10.80 | 14.22 | 18.00 | 22.86 | 28.08 | 40.50 | 72.00 |
| 19 | 2.09 | 3.80 | 5.89 | 8.36 | 11.40 | 15.01 | 19.00 | 24.13 | 29.64 | 42.75 | 76.00 |
| 20 | 2.20 | 4.00 | 6.20 | 8.80 | 12.00 | 15.80 | 20.00 | 25.40 | 31.20 | 45.00 | 80.00 |
| 21 | 2.31 | 4.20 | 6.51 | 9.24 | 12.60 | 16.59 | 21.00 | 26.67 | 32.76 | 47.25 | 84.00 |
| 22 | 2.42 | 4.40 | 6.82 | 9.68 | 13.20 | 17.38 | 22.00 | 27.94 | 34.32 | 49.50 | 88.00 |
| 23 | 2.53 | 4.60 | 7.13 | 10.12 | 13.80 | 18.17 | 23.00 | 29.21 | 35.88 | 51.75 | 92.00 |
| 24 | 2.64 | 4.80 | 7.44 | 10.56 | 14.40 | 18.96 | 24.00 | 30.48 | 37.44 | 54.00 | 96.00 |
| 25 | 2.75 | 5.00 | 7.75 | 11.00 | 15.00 | 19.75 | 25.00 | 31.75 | 39.00 | 56.25 | 100.00 |
| 26 | 2.86 | 5.20 | 8.06 | 11.44 | 15.60 | 20.54 | 26.00 | 33.02 | 40.56 | 58.50 | 104.00 |
| 27 | 2.97 | 5.40 | 8.37 | 11.88 | 16.20 | 21.33 | 27.00 | 34.29 | 42.12 | 60.75 | 108.00 |
| 28 | 3.08 | 5.60 | 8.68 | 12.32 | 16.80 | 22.12 | 28.00 | 35.56 | 43.68 | 63.00 | 112.00 |
| 29 | 3.19 | 5.80 | 8.99 | 12.76 | 17.40 | 22.91 | 29.00 | 36.83 | 45.24 | 65.25 | 116.00 |
| 30 | 3.30 | 6.00 | 9.30 | 13.20 | 18.00 | 23.70 | 30.00 | 38.10 | 46.80 | 67.50 | 120.00 |

Areas for Various Bar Sizes and Number of Bars
Table 5.1-A3

## Tension Development Length of Uncoated Deformed Bars

| | $f_c'$ = 3,000 psi | | $f_c'$ = 4,000 psi | | $f_c'$ = 5,000 psi | | $f_c'$ = 6,000 psi | |
|---|---|---|---|---|---|---|---|---|
| Bar Size | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in |
| 3 | 1′-5″ | 1′-0″ | 1′-5″ | 1′-0″ | 1′-5″ | 1′-0″ | 1′-5″ | 1′-0″ |
| 4 | 1′-5″ | 1′-0″ | 1′-5″ | 1′-0″ | 1′-5″ | 1′-0″ | 1′-5″ | 1′-0″ |
| 5 | 1′-9″ | 1′-3″ | 1′-9″ | 1′-3″ | 1′-9″ | 1′-3″ | 1′-9″ | 1′-3″ |
| 6 | 2′-3″ | 1′-8″ | 2′-2″ | 1′-6″ | 2′-2″ | 1′-6″ | 2′-2″ | 1′-6″ |
| 7 | 3′-1″ | 2′-3″ | 2′-8″ | 1′-11″ | 2′-6″ | 1′-9″ | 2′-6″ | 1′-9″ |
| 8 | 4′-1″ | 2′-11″ | 3′-6″ | 2′-6″ | 3′-2″ | 2′-3″ | 2′-11″ | 2′-1″ |
| 9 | 5′-2″ | 3′-8″ | 4′-6″ | 3′-2″ | 4′-0″ | 2′-10″ | 3′-8″ | 2′-7″ |
| 10 | 6′-6″ | 4′-8″ | 5′-8″ | 4′-1″ | 5′-1″ | 3′-8″ | 4′-8″ | 3′-4″ |
| 11 | 8′-0″ | 5′-9″ | 6′-11″ | 5′-0″ | 6′-3″ | 4′-5″ | 5′-8″ | 4′-1″ |
| 14 | 10′-11″ | 7′-10″ | 9′-5″ | 6′-9″ | 8′-5″ | 6′-1″ | 7′-9″ | 5′-6″ |
| 18 | 14′-1″ | 10′-1″ | 12′-3″ | 8′-9″ | 10′-11″ | 7′-10″ | 10′-0″ | 7′-2″ |

## Tension Development Length of Epoxy Coated Deformed Bars

| | $f_c'$ = 3,000 psi | | $f_c'$ = 4,000 psi | | $f_c'$ = 5,000 psi | | $f_c'$ = 6,000 psi | |
|---|---|---|---|---|---|---|---|---|
| Bar Size | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in |
| 3 | 1′-9″ | 1′-6″ | 1′-9″ | 1′-6″ | 1′-9″ | 1′-6″ | 1′-9″ | 1′-6″ |
| 4 | 1′-9″ | 1′-6″ | 1′-9″ | 1′-6″ | 1′-9″ | 1′-6″ | 1′-9″ | 1′-6″ |
| 5 | 2′-2″ | 1′-11″ | 2′-2″ | 1′-11″ | 2′-2″ | 1′-11″ | 2′-2″ | 1′-11″ |
| 6 | 2′-9″ | 2′-5″ | 2′-7″ | 2′-3″ | 2′-7″ | 2′-3″ | 2′-7″ | 2′-3″ |
| 7 | 3′-9″ | 3′-4″ | 3′-3″ | 2′-11″ | 3′-0″ | 2′-8″ | 3′-0″ | 2′-8″ |
| 8 | 4′-11″ | 4′-4″ | 4′-3″ | 3′-9″ | 3′-10″ | 3′-5″ | 3′-6″ | 3′-1″ |
| 9 | 6′-3″ | 5′-6″ | 5′-5″ | 4′-9″ | 4′-10″ | 4′-3″ | 4′-5″ | 3′-11″ |
| 10 | 7′-11″ | 7′-0″ | 6′-10″ | 6′-1″ | 6′-2″ | 5′-5″ | 5′-7″ | 4′-11″ |
| 11 | 9′-9″ | 8′-7″ | 8′-5″ | 7′-5″ | 7′-6″ | 6′-8″ | 6′-11″ | 6′-1″ |
| 14 | 13′-3″ | 11′-8″ | 11′-6″ | 10′-1″ | 10′-3″ | 9′-1″ | 9′-4″ | 8′-3″ |
| 18 | 17′-1″ | 15′-1″ | 14′-10″ | 13′-1″ | 13′-3″ | 11′-8″ | 12′-1″ | 10′-8″ |

Top Bars are so placed that more than 12″ of concrete is cast below the reinforcement. Modification Factor for Spacing >=6″ and side cover >=3″ = 0.8. Minimum Development Length = 12″. Modification Factor for Reinforcement Enclosed in Spiral = 0.75

Table 5.1-A4

## Tension Development Length of Standard 90° and 180° Hooks

| | $f_c'$ = 3,000 psi | | $f_c'$ = 4,000 psi | | $f_c'$ = 5,000 psi | | $f_c'$ = 6,000 psi | |
|---|---|---|---|---|---|---|---|---|
| Bar Size | Side Cover < 2½″ Cover on Tail < 2″ | Side Cover >= 2½″ Cover on Tail >= 2″ | Side Cover < 2½″ Cover on Tail < 2″ | Side Cover >= 2½″ Cover on Tail >= 2″ | Side Cover < 2½″ Cover on Tail < 2″ | Side Cover >= 2½″ Cover on Tail >= 2″ | Side Cover < 2½″ Cover on Tail < 2″ | Side Cover >= 2½″ Cover on Tail >= 2″ |
| 3 | 0′-9″ | 0′-6″ | 0′-8″ | 0′-6″ | 0′-7″ | 0′-6″ | 0′-6″ | 0′-6″ |
| 4 | 0′-11″ | 0′-8″ | 0′-10″ | 0′-7″ | 0′-9″ | 0′-7″ | 0′-8″ | 0′-7″ |
| 5 | 1′-2″ | 0′-10″ | 1′-0″ | 0′-9″ | 0′-11″ | 0′-8″ | 0′-10″ | 0′-7″ |
| 6 | 1′-5″ | 1′-0″ | 1′-3″ | 0′-10″ | 1′-1″ | 0′-9″ | 1′-0″ | 0′-8″ |
| 7 | 1′-8″ | 1′-2″ | 1′-5″ | 1′-0″ | 1′-3″ | 0′-11″ | 1′-2″ | 0′-10″ |
| 8 | 1′-10″ | 1′-4″ | 1′-7″ | 1′-2″ | 1′-5″ | 1′-0″ | 1′-4″ | 0′-11″ |
| 9 | 2′-1″ | 1′-6″ | 1′-10″ | 1′-3″ | 1′-8″ | 1′-2″ | 1′-6″ | 1′-1″ |
| 10 | 2′-4″ | 1′-8″ | 2′-1″ | 1′-5″ | 1′-10″ | 1′-3″ | 1′-8″ | 1′-2″ |
| 11 | 2′-7″ | 1′-10″ | 2′-3″ | 1′-7″ | 2′-0″ | 1′-5″ | 1′-10″ | 1′-4″ |
| 14 | 3′-1″ | 3′-1″ | 2′-9″ | 2′-9″ | 2′-5″ | 2′-5″ | 2′-3″ | 2′-3″ |
| 18 | 4′-2″ | 4′-2″ | 3′-7″ | 3′-7″ | 3′-3″ | 3′-3″ | 2′-11″ | 2′-11″ |

Table 5.1-A5

### Tension Lap Splice Lengths of Grade 60 Uncoated Bars – Class B

| | $f_c'$ = 3,000 psi | | $f_c'$ = 4,000 psi | | $f_c'$ = 5,000 psi | | $f_c'$ = 6,000 psi | |
|---|---|---|---|---|---|---|---|---|
| Bar Size | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in |
| 3 | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ |
| 4 | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ | 2′-0″ |
| 5 | 2′-4″ | 2′-0″ | 2′-4″ | 2′-0″ | 2′-4″ | 2′-0″ | 2′-4″ | 2′-0″ |
| 6 | 2′-11″ | 2′-1″ | 2′-9″ | 2′-0″ | 2′-9″ | 2′-0″ | 2′-9″ | 2′-0″ |
| 7 | 4′-0″ | 2′-11″ | 3′-6″ | 2′-6″ | 3′-3″ | 2′-4″ | 3′-3″ | 2′-4″ |
| 8 | 5′-3″ | 3′-9″ | 4′-7″ | 3′-3″ | 4′-11″ | 2′-11″ | 3′-9″ | 2′-8″ |
| 9 | 6′-8″ | 4′-9″ | 5′-9″ | 4′-2″ | 5′-2″ | 3′-9″ | 4′-9″ | 3′-5″ |
| 10 | 8′-6″ | 6′-1″ | 7′-4″ | 5′-3″ | 6′-7″ | 4′-8″ | 6′-0″ | 4′-4″ |
| 11 | 10′-5″ | 7′-5″ | 9′-0″ | 6′-5″ | 8′-1″ | 5′-9″ | 7′-4″ | 5′-3″ |
| 14 | Lap Splices | | Lap Splices | | Lap Splices | | Lap Splices | |
| 18 | Not Allowed | | Not Allowed | | Not Allowed | | Not Allowed | |

### Tension Lap Splice Lengths of Grade 60 Epoxy Coated Bars – Class B

| | $f_c'$ = 3,000 psi | | $f_c'$ = 4,000 psi | | $f_c'$ = 5,000 psi | | $f_c'$ = 6,000 psi | |
|---|---|---|---|---|---|---|---|---|
| Bar Size | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in | Top Bars ft-in | Others ft-in |
| 3 | 2′-3″ | 2′-0″ | 2′-3″ | 2′-0″ | 2′-3″ | 2′-0″ | 2′-3″ | 2′-0″ |
| 4 | 2′-3″ | 2′-0″ | 2′-3″ | 2′-0″ | 2′-3″ | 2′-0″ | 2′-3″ | 2′-0″ |
| 5 | 2′-10″ | 2′-6″ | 2′-10″ | 2′-6″ | 2′-10″ | 2′-6″ | 2′-10″ | 2′-6″ |
| 6 | 3′-7″ | 3′-2″ | 3′-4″ | 3′-0″ | 3′-4″ | 3′-0″ | 3′-4″ | 3′-0″ |
| 7 | 4′-11″ | 4′-4″ | 4′-3″ | 3′-9″ | 3′-11″ | 3′-5″ | 3′-11″ | 3′-5″ |
| 8 | 6′-5″ | 5′-8″ | 5′-7″ | 4′-11″ | 5′-0″ | 4′-5″ | 4′-6″ | 4′-0″ |
| 9 | 8′-1″ | 7′-2″ | 7′-0″ | 6′-2″ | 6′-3″ | 5′-7″ | 5′-9″ | 5′-1″ |
| 10 | 10′-3″ | 9′-1″ | 8′-11″ | 7′-10″ | 8′-0″ | 7′-0″ | 7′-3″ | 6′-5″ |
| 11 | 12′-8″ | 11′-2″ | 10′-11″ | 9′-8″ | 9′-9″ | 8′-0″ | 8′-11″ | 7′-11″ |
| 14 | Lap Splices | | Lap Splices | | Lap Splices | | Lap Splices | |
| 18 | Not Allowed | | Not Allowed | | Not Allowed | | Not Allowed | |

Top Bars are so placed that more than 12″ of concrete is cast below the reinforcement.

Definition of Splice Classes:
- Class A: Low stressed bars – 75% or less are spliced
- Class B: Low stressed bars – more than 75% are spliced; High stressed bars – 1/2 or less are spliced
- Class C: High stressed bars – more than 50% are spliced

Class B Lap splice is the preferred and most commonly used by bridge office.

Modification Factor for Class A: 0.77
Modification Factor for Class C: 1.31
Modification Factor for 3-bar Bundle = 1.2

Table 5.1-A6

**Development Length of Deformed Bars in Compression and Minimum Compression Lap Splice**

Per AASHTO Standard Specifications, 1991, 16th Edition Articles 8.26, 8.32.4

| Concrete | $f_c'$ = 3,000 psi | $f_c'$ = 4,000 psi | $f_c'$ = 5,000 psi | $f_c'$ = 6,000 psi | $f_c'$ > 3,000 psi |
|---|---|---|---|---|---|
| Reinf. | Grade 60 | Grade 60 | Grade 60 | Grade 60 | Grade 60 |
| Bar Size | Development Length, $l_d$ | | | | Minimum Lap Splice |
| 3 & 4 | 1′-0″* | 1′-0″* | 1′-0″* | 1′-0″* | 2′-0″4 |
| 5 | 1′-2″ | 1′-0″ | 1′-0″* | 1′-0″* | 2′-0″4 |
| 6 | 1′-5″ | 1′-3″ | 1′-2″ | 1′-2″ | 2′-0″4 |
| 7 | 1′-8″ | 1′-5″ | 1′-4″ | 1′-4″ | 2′-3″ |
| 8 | 1′-10″ | 1′-7″ | 1′-6″ | 1′-6″ | 2′-6″ |
| 9 | 2′-1″ | 1′-10″ | 1′-9″ | 1′-9″ | 2′-10″ |
| 10 | 2′-4″ | 2′-1″ | 1′-11″ | 1′-11″ | 3′-3″ |
| 11 | 2′-7″ | 2′-3″ | 2′-2″ | 2′-2″ | 3′-7″ |
| 14 | 3′-1″ | 2′-9″ | 2′-7″ | 2′-7″ | 4′-3″ |
| 18 | 4′-2″ | 3′-7″ | 3′-5″ | 3′-5″ | 5′-8″ |

*Note:*

1. Where excess bar area is provided, $l_d$ may be reduced by the ratio of required area to area provided.
2. *1′-0″ minimum (office practice).
3. $l_d$ (compression) must be developed with straight bar extension. Reduced length noted in (1) shall also be straight bar extension.
4. 2′-0″ minimum (office practice).
5. When splicing smaller bars to larger bars, the lap splice shall be the larger of the minimum compression lap splice or the development length of the larger bar in compression, AASHTO Art. 8.32.4.1.

Table 5.1-A7

| ρ | $\frac{M_u}{\phi bd^2}$ | ρ | $\frac{M_u}{\phi bd^2}$ | ρ | $\frac{M_u}{\phi bd^2}$ | ρ | $\frac{M_u}{\phi bd^2}$ |
|---|---|---|---|---|---|---|---|
| 0.0010 | 59.3 | 0.0053 | 298.1 | 0.0097 | 515.4 | 0.0141 | 705.2 |
| 0.0011 | 65.1 | 0.0054 | 303.4 | 0.0098 | 520.0 | 0.0142 | 709.2 |
| 0.0012 | 71.0 | 0.0055 | 308.6 | 0.0099 | 524.6 | 0.0143 | 713.2 |
| 0.0013 | 76.8 | 0.0056 | 313.8 | 0.0100 | 529.2 | 0.0144 | 717.2 |
| 0.0014 | 82.6 | 0.0057 | 319.0 | 0.0101 | 533.8 | 0.0145 | 721.1 |
| 0.0015 | 88.4 | 0.0058 | 324.2 | 0.0102 | 538.3 | 0.0146 | 725.1 |
| 0.0016 | 94.2 | 0.0059 | 329.4 | 0.0103 | 542.9 | 0.0147 | 729.0 |
| 0.0017 | 100.0 | 0.0060 | 334.5 | 0.0104 | 547.4 | 0.0148 | 732.9 |
| 0.0018 | 105.7 | 0.0061 | 339.7 | 0.0105 | 551.9 | 0.0149 | 736.8 |
| 0.0019 | 111.4 | 0.0062 | 344.8 | 0.0106 | 556.4 | 0.0150 | 740.7 |
| 0.0020 | 117.2 | 0.0063 | 349.9 | 0.0107 | 560.9 | 0.0151 | 744.6 |
| 0.0021 | 122.9 | 0.0064 | 355.0 | 0.0108 | 565.4 | 0.0152 | 748.4 |
| 0.0022 | 128.6 | 0.0065 | 360.1 | 0.0109 | 569.9 | 0.0153 | 752.3 |
| 0.0023 | 134.3 | 0.0066 | 365.2 | 0.0110 | 574.3 | 0.0154 | 756.1 |
| 0.0024 | 139.9 | 0.0067 | 370.2 | 0.0111 | 578.8 | 0.0155 | 759.9 |
| 0.0025 | 145.6 | 0.0068 | 375.3 | 0.0112 | 583.2 | 0.0156 | 763.7 |
| 0.0026 | 151.2 | 0.0069 | 380.3 | 0.0113 | 587.6 | 0.0157 | 767.5 |
| 0.0027 | 156.8 | 0.0070 | 385.3 | 0.0114 | 592.0 | 0.0158 | 771.2 |
| 0.0028 | 162.4 | 0.0071 | 390.3 | 0.0115 | 596.4 | 0.0159 | 775.0 |
| 0.0029 | 168.0 | 0.0072 | 395.0 | 0.0116 | 600.7 | 0.0160 | 778.7 |
| 0.0030 | 173.6 | 0.0073 | 400.3 | 0.0117 | 605.1 | $\rho_{max}$ 0.0161 | 782.5 |
| 0.0031 | 179.2 | 0.0074 | 405.2 | 0.0118 | 609.4 | | |
| 0.0032 | 184.8 | 0.0075 | 410.2 | 0.0119 | 613.7 | | |
| 0.0033 | 190.3 | 0.0076 | 415.1 | 0.0120 | 618.0 | | |
| 0.0034 | 195.8 | 0.0077 | 420.0 | 0.0121 | 622.3 | | |
| 0.0035 | 201.3 | 0.0078 | 424.9 | 0.0122 | 626.6 | | |
| 0.0036 | 206.8 | 0.0079 | 429.8 | 0.0123 | 630.9 | | |
| 0.0037 | 212.3 | 0.0080 | 434.7 | 0.0124 | 635.1 | | |
| 0.0038 | 217.8 | 0.0081 | 439.5 | 0.0125 | 639.4 | | |
| 0.0039 | 223.2 | 0.0082 | 444.4 | 0.0126 | 643.6 | | |
| 0.0040 | 228.7 | 0.0083 | 449.2 | 0.0127 | 647.8 | | |
| 0.0041 | 234.1 | 0.0084 | 454.0 | 0.0128 | 652.0 | | |
| 0.0042 | 239.5 | 0.0085 | 458.8 | 0.0129 | 656.2 | | |
| 0.0043 | 244.9 | 0.0086 | 463.6 | 0.0130 | 660.3 | | |
| 0.0044 | 250.3 | 0.0087 | 468.4 | 0.0131 | 664.5 | | |
| 0.0045 | 255.7 | 0.0088 | 473.2 | 0.0132 | 668.6 | | |
| 0.0046 | 261.0 | 0.0089 | 477.9 | 0.0133 | 672.8 | | |
| 0.0047 | 266.4 | 0.0090 | 482.6 | 0.0134 | 676.9 | | |
| 0.0048 | 271.7 | 0.0091 | 487.4 | 0.0135 | 681.0 | | |
| 0.0049 | 277.0 | 0.0092 | 492.1 | 0.0136 | 685.0 | | |
| 0.0050 | 282.3 | 0.0093 | 496.8 | 0.0137 | 689.1 | | |
| 0.0051 | 287.6 | 0.0094 | 501.4 | 0.0138 | 693.2 | | |
| 0.0052 | 292.9 | 0.0095 | 506.1 | 0.0139 | 697.2 | | |
| | | 0.0096 | 510.7 | 0.0140 | 701.2 | | |

**Notes:**

1. Units of $\frac{M_u}{\rho bd^2}$ are in psi.
2. $\rho_{min}$ should be based on 1.2 Mcr or 1.33 ρ analysis, whichever is smaller.
3. $\rho_{max} = 0.75\rho_b = 0.0161$ based on $\beta_1 = 0.85$.

Table 5.2-A1

| ρ | $\frac{M_u}{\phi bd^2}$ | ρ | $\frac{M_u}{\phi bd^2}$ | ρ | $\frac{M_u}{\phi bd^2}$ | ρ | $\frac{M_u}{\phi bd^2}$ | ρ | $\frac{M_u}{\phi bd^2}$ |
|---|---|---|---|---|---|---|---|---|---|
| 0.0010 | 59.5 | 0.0056 | 319.3 | 0.0102 | 556.7 | 0.0148 | 771.7 | 0.0194 | 964.1 |
| 0.0011 | 65.4 | 0.0057 | 324.7 | 0.0103 | 561.7 | 0.0149 | 776.1 | 0.0195 | 968.1 |
| 0.0012 | 71.2 | 0.0058 | 330.1 | 0.0104 | 566.6 | 0.0150 | 780.5 | 0.0196 | 972.0 |
| 0.0013 | 77.1 | 0.0059 | 335.5 | 0.0105 | 571.5 | 0.0151 | 784.9 | 0.0197 | 975.9 |
| 0.0014 | 83.0 | 0.0060 | 340.9 | 0.0106 | 576.3 | 0.0152 | 789.3 | 0.0198 | 979.8 |
| 0.0015 | 88.8 | 0.0061 | 346.2 | 0.0107 | 581.2 | 0.0153 | 793.7 | 0.0199 | 983.7 |
| 0.0016 | 94.6 | 0.0062 | 351.6 | 0.0108 | 586.1 | 0.0154 | 798.1 | 0.0200 | 987.6 |
| 0.0017 | 100.5 | 0.0063 | 356.9 | 0.0109 | 590.9 | 0.0155 | 802.4 | 0.0201 | 991.5 |
| 0.0018 | 106.3 | 0.0064 | 362.2 | 0.0110 | 595.7 | 0.0156 | 806.8 | 0.0202 | 995.3 |
| 0.0019 | 112.1 | 0.0065 | 367.6 | 0.0111 | 600.6 | 0.0157 | 811.1 | 0.0203 | 999.2 |
| 0.0020 | 117.9 | 0.0066 | 372.9 | 0.0112 | 605.4 | 0.0158 | 815.4 | 0.0204 | 1003.0 |
| 0.0021 | 123.7 | 0.0067 | 378.2 | 0.0113 | 610.2 | 0.0159 | 819.7 | 0.0205 | 1006.8 |
| 0.0022 | 129.4 | 0.0068 | 383.4 | 0.0114 | 615.0 | 0.0160 | 824.1 | 0.0206 | 1010.7 |
| 0.0023 | 135.2 | 0.0069 | 388.7 | 0.0115 | 619.8 | 0.0161 | 828.3 | 0.0207 | 1014.5 |
| 0.0024 | 140.9 | 0.0070 | 394.0 | 0.0116 | 624.5 | 0.0162 | 832.6 | 0.0208 | 1018.3 |
| 0.0025 | 146.7 | 0.0071 | 399.2 | 0.0117 | 629.3 | 0.0163 | 836.9 | 0.0209 | 1022.0 |
| 0.0026 | 152.4 | 0.0072 | 404.5 | 0.0118 | 634.1 | 0.0164 | 841.2 | 0.0210 | 1025.8 |
| 0.0027 | 158.1 | 0.0073 | 409.7 | 0.0119 | 638.8 | 0.0165 | 845.4 | 0.0211 | 1029.6 |
| 0.0028 | 163.8 | 0.0074 | 414.9 | 0.0120 | 643.5 | 0.0166 | 849.7 | 0.0212 | 1033.3 |
| 0.0029 | 169.5 | 0.0075 | 420.1 | 0.0121 | 648.2 | 0.0167 | 853.9 | 0.0213 | 1037.1 |
| 0.0030 | 175.2 | 0.0076 | 425.3 | 0.0122 | 653.0 | 0.0168 | 858.1 | $\rho_{max}$ 0.0214 | 1040.8 |
| 0.0031 | 180.9 | 0.0077 | 430.5 | 0.0123 | 657.7 | 0.0169 | 862.3 | | |
| 0.0032 | 186.6 | 0.0078 | 435.7 | 0.0124 | 662.3 | 0.0170 | 866.5 | | |
| 0.0033 | 192.2 | 0.0079 | 440.9 | 0.0125 | 667.0 | 0.0171 | 870.7 | | |
| 0.0034 | 197.9 | 0.0080 | 446.0 | 0.0126 | 671.7 | 0.0172 | 874.9 | | |
| 0.0035 | 203.5 | 0.0081 | 451.2 | 0.0127 | 676.3 | 0.0173 | 879.1 | | |
| 0.0036 | 209.1 | 0.0082 | 456.3 | 0.0128 | 681.0 | 0.0174 | 883.2 | | |
| 0.0037 | 214.7 | 0.0083 | 461.4 | 0.0129 | 685.6 | 0.0175 | 887.4 | | |
| 0.0038 | 220.3 | 0.0084 | 466.5 | 0.0130 | 690.3 | 0.0176 | 891.5 | | |
| 0.0039 | 225.9 | 0.0085 | 471.6 | 0.0131 | 694.9 | 0.0177 | 895.6 | | |
| 0.0040 | 231.5 | 0.0086 | 476.7 | 0.0132 | 699.5 | 0.0178 | 899.7 | | |
| 0.0041 | 237.1 | 0.0087 | 481.8 | 0.0133 | 704.1 | 0.0179 | 903.9 | | |
| 0.0042 | 242.6 | 0.0088 | 486.9 | 0.0134 | 708.6 | 0.0180 | 907.9 | | |
| 0.0043 | 248.2 | 0.0089 | 491.9 | 0.0135 | 713.2 | 0.0181 | 912.0 | | |
| 0.0044 | 253.7 | 0.0090 | 497.0 | 0.0136 | 717.8 | 0.0182 | 916.1 | | |
| 0.0045 | 259.2 | 0.0091 | 502.0 | 0.0137 | 722.3 | 0.0183 | 920.2 | | |
| 0.0046 | 264.8 | 0.0092 | 507.1 | 0.0138 | 726.9 | 0.0184 | 924.2 | | |
| 0.0047 | 270.3 | 0.0093 | 512.1 | 0.0139 | 731.4 | 0.0185 | 928.3 | | |
| 0.0048 | 275.8 | 0.0094 | 517.1 | 0.0140 | 735.9 | 0.0186 | 932.3 | | |
| 0.0049 | 281.2 | 0.0095 | 522.1 | 0.0141 | 740.4 | 0.0187 | 936.3 | | |
| 0.0050 | 286.7 | 0.0096 | 527.1 | 0.0142 | 744.9 | 0.0188 | 940.3 | | |
| 0.0051 | 292.2 | 0.0097 | 532.0 | 0.0143 | 749.4 | 0.0189 | 944.3 | | |
| 0.0052 | 297.6 | 0.0098 | 537.0 | 0.0144 | 753.9 | 0.0190 | 948.3 | | |
| 0.0053 | 303.1 | 0.0099 | 542.0 | 0.0145 | 758.3 | 0.0191 | 952.3 | | |
| 0.0054 | 308.5 | 0.0100 | 546.9 | 0.0146 | 762.8 | 0.0192 | 956.2 | | |
| 0.0055 | 313.9 | 0.0101 | 551.8 | 0.0147 | 767.2 | 0.0193 | 960.2 | | |

**Notes:**

1. Units of $\frac{M_u}{\rho bd^2}$ are in psi.
2. $\rho_{min}$ should be based on 1.2 Mcr or 1.33 ρ analysis, whichever is smaller.
3. $\rho_{max} = 0.75\rho_b = 0.0214$ based on $\beta_1 = 0.85$.

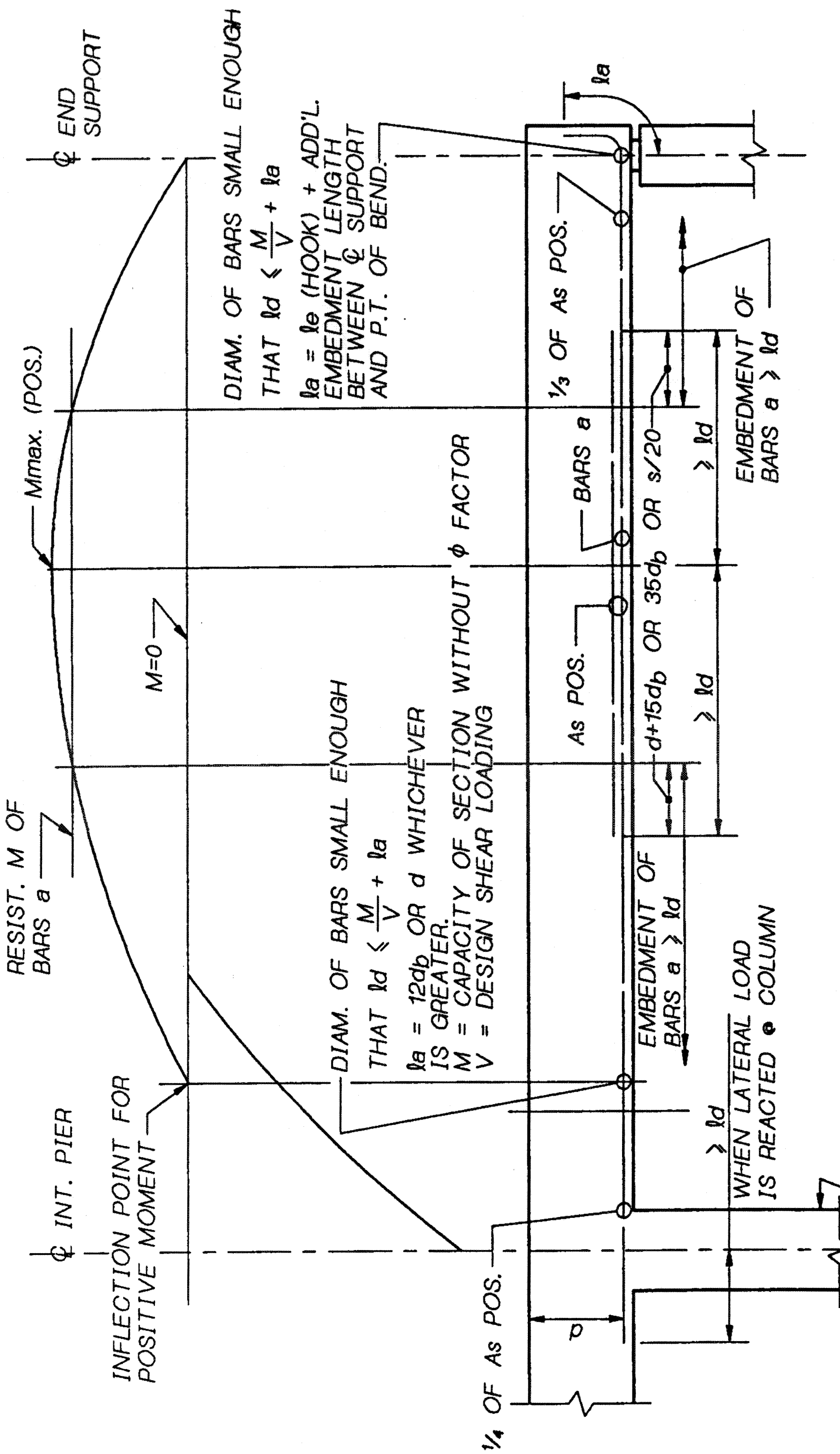
Table 5.2-A2

| $\rho$ | $\frac{M_u}{\phi bd^2}$ | $\rho$ | $\frac{M_u}{\phi bd^2}$ | $\rho$ | $\frac{M_u}{\phi bd^2}$ | $\rho$ | $\frac{M_u}{\phi bd^2}$ | $\rho$ | $\frac{M_u}{\phi bd^2}$ |
|---|---|---|---|---|---|---|---|---|---|
| 0.0010 | 59.6 | 0.0061 | 350.2 | 0.0113 | 623.8 | 0.0165 | 874.3 | 0.0217 | 1102.0 |
| 0.0011 | 65.5 | 0.0062 | 355.7 | 0.0114 | 628.8 | 0.0166 | 878.9 | 0.0218 | 1106.1 |
| 0.0012 | 71.4 | 0.0063 | 361.1 | 0.0115 | 633.8 | 0.0167 | 883.5 | 0.0219 | 1110.3 |
| 0.0013 | 77.3 | 0.0064 | 366.6 | 0.0116 | 638.8 | 0.0168 | 888.1 | 0.0220 | 1114.4 |
| 0.0014 | 83.2 | 0.0065 | 372.1 | 0.0117 | 643.8 | 0.0169 | 892.7 | 0.0221 | 1118.5 |
| 0.0015 | 89.0 | 0.0066 | 377.5 | 0.0118 | 648.9 | 0.0170 | 897.2 | 0.0222 | 1122.6 |
| 0.0016 | 94.9 | 0.0067 | 382.9 | 0.0119 | 653.8 | 0.0171 | 901.8 | 0.0223 | 1126.8 |
| 0.0017 | 100.8 | 0.0068 | 388.4 | 0.0120 | 658.8 | 0.0172 | 906.3 | 0.0224 | 1130.9 |
| 0.0018 | 106.6 | 0.0069 | 393.8 | 0.0121 | 663.8 | 0.0173 | 910.9 | 0.0225 | 1134.9 |
| 0.0019 | 112.5 | 0.0070 | 399.2 | 0.0122 | 668.8 | 0.0174 | 915.4 | 0.0226 | 1139.0 |
| 0.0020 | 118.3 | 0.0071 | 404.6 | 0.0123 | 673.7 | 0.0175 | 919.9 | 0.0227 | 1143.1 |
| 0.0021 | 124.1 | 0.0072 | 410.0 | 0.0124 | 678.7 | 0.0176 | 924.4 | 0.0228 | 1147.2 |
| 0.0022 | 129.9 | 0.0073 | 415.4 | 0.0125 | 683.6 | 0.0177 | 928.9 | 0.0229 | 1151.2 |
| 0.0023 | 135.8 | 0.0074 | 420.7 | 0.0126 | 688.6 | 0.0178 | 933.4 | 0.0230 | 1155.3 |
| 0.0024 | 141.6 | 0.0075 | 426.1 | 0.0127 | 693.5 | 0.0179 | 937.9 | 0.0231 | 1159.3 |
| 0.0025 | 147.3 | 0.0076 | 431.5 | 0.0128 | 698.4 | 0.0180 | 942.4 | 0.0232 | 1163.4 |
| 0.0026 | 153.1 | 0.0077 | 436.8 | 0.0129 | 703.3 | 0.0181 | 946.8 | 0.0233 | 1167.4 |
| 0.0027 | 158.9 | 0.0078 | 442.2 | 0.0130 | 708.2 | 0.0182 | 951.3 | 0.0234 | 1171.4 |
| 0.0028 | 164.7 | 0.0079 | 447.5 | 0.0131 | 713.1 | 0.0183 | 955.7 | 0.0235 | 1175.4 |
| 0.0029 | 170.4 | 0.0080 | 452.8 | 0.0132 | 718.0 | 0.0184 | 960.2 | 0.0236 | 1179.4 |
| 0.0030 | 176.2 | 0.0081 | 458.1 | 0.0133 | 722.9 | 0.0185 | 964.6 | 0.0237 | 1183.4 |
| 0.0031 | 181.9 | 0.0082 | 463.4 | 0.0134 | 727.7 | 0.0186 | 969.0 | 0.0238 | 1187.4 |
| 0.0032 | 187.7 | 0.0083 | 468.7 | 0.0135 | 732.6 | 0.0187 | 973.5 | 0.0239 | 1191.4 |
| 0.0033 | 193.4 | 0.0084 | 474.0 | 0.0136 | 737.4 | 0.0188 | 977.9 | 0.0240 | 1195.3 |
| 0.0034 | 199.1 | 0.0085 | 479.3 | 0.0137 | 742.3 | 0.0189 | 982.3 | 0.0241 | 1199.3 |
| 0.0035 | 204.8 | 0.0086 | 484.6 | 0.0138 | 747.1 | 0.0190 | 986.6 | 0.0242 | 1203.2 |
| 0.0036 | 210.5 | 0.0087 | 489.8 | 0.0139 | 751.9 | 0.0191 | 991.0 | 0.0243 | 1207.2 |
| 0.0037 | 216.2 | 0.0088 | 495.1 | 0.0140 | 756.7 | 0.0192 | 995.4 | 0.0244 | 1211.1 |
| 0.0038 | 221.9 | 0.0089 | 500.4 | 0.0141 | 761.5 | 0.0193 | 999.8 | 0.0245 | 1215.0 |
| 0.0039 | 227.5 | 0.0090 | 505.6 | 0.0142 | 766.3 | 0.0194 | 1004.1 | 0.0246 | 1218.9 |
| 0.0040 | 233.2 | 0.0091 | 510.8 | 0.0143 | 771.1 | 0.0195 | 1008.5 | 0.0247 | 1222.8 |
| 0.0041 | 238.9 | 0.0092 | 516.0 | 0.0144 | 775.9 | 0.0196 | 1012.8 | 0.0248 | 1226.7 |
| 0.0042 | 244.5 | 0.0093 | 521.3 | 0.0145 | 780.7 | 0.0197 | 1017.1 | 0.0249 | 1230.6 |
| 0.0043 | 250.1 | 0.0094 | 526.5 | 0.0146 | 785.4 | 0.0198 | 1021.5 | 0.0250 | 1234.5 |
| 0.0044 | 255.8 | 0.0095 | 531.7 | 0.0147 | 790.2 | 0.0199 | 1025.8 | 0.0251 | 1238.4 |
| 0.0045 | 261.4 | 0.0096 | 536.9 | 0.0148 | 795.0 | 0.0200 | 1030.1 | $\rho_{max}$ 0.0252 | 1242.2 |
| 0.0046 | 267.0 | 0.0097 | 542.0 | 0.0149 | 799.7 | 0.0201 | 1034.4 | | |
| 0.0047 | 272.6 | 0.0098 | 547.2 | 0.0150 | 804.4 | 0.0202 | 1038.7 | | |
| 0.0048 | 278.2 | 0.0099 | 552.4 | 0.0151 | 809.1 | 0.0203 | 1042.9 | | |
| 0.0049 | 283.8 | 0.0100 | 557.5 | 0.0152 | 813.9 | 0.0204 | 1047.2 | | |
| 0.0050 | 289.4 | 0.0101 | 562.7 | 0.0153 | 818.6 | 0.0205 | 1051.5 | | |
| 0.0051 | 295.0 | 0.0102 | 567.8 | 0.0154 | 823.3 | 0.0206 | 1055.7 | | |
| 0.0052 | 300.5 | 0.0103 | 572.9 | 0.0155 | 827.9 | 0.0207 | 1060.0 | | |
| 0.0053 | 306.1 | 0.0104 | 578.1 | 0.0156 | 832.6 | 0.0208 | 1064.2 | | |
| 0.0054 | 311.6 | 0.0105 | 583.2 | 0.0157 | 837.3 | 0.0209 | 1068.4 | | |
| 0.0055 | 317.1 | 0.0106 | 588.3 | 0.0158 | 842.0 | 0.0210 | 1072.7 | | |
| 0.0056 | 322.7 | 0.0107 | 593.4 | 0.0159 | 846.6 | 0.0211 | 1076.9 | | |
| 0.0057 | 328.2 | 0.0108 | 598.5 | 0.0160 | 851.3 | 0.0212 | 1081.1 | | |
| 0.0058 | 333.7 | 0.0109 | 603.5 | 0.0161 | 855.9 | 0.0213 | 1085.3 | | |
| 0.0059 | 339.2 | 0.0110 | 608.6 | 0.0162 | 860.5 | 0.0214 | 1089.5 | | |
| 0.0060 | 344.7 | 0.0111 | 613.7 | 0.0163 | 865.1 | 0.0215 | 1093.6 | | |
| | | 0.0112 | 618.7 | 0.0164 | 869.7 | 0.0216 | 1097.8 | | |

**Notes:**

1. Units of $\frac{M_u}{\rho bd^2}$ are in psi.
2. $\rho_{min}$ should be based on 1.2 Mcr or 1.33 $\rho$ analysis, whichever is smaller.
3. $\rho_{max} = 0.75\rho_b = 0.0252$ based on $\beta_1 = 0.80$.

Table 5.2-A3




TYPICAL END SPAN

ALSO CHECK: AASHTO 8.17.2 - DISTRIBUTION OF REINF. IN FLEXURAL MEMBERS.
AASHTO 8.16.8.3 - FATIGUE STRESS LIMITS.
AASHTO 8.16.8.4 . DISTRIBUTION OF FLEXURAL REINFORCEMENT.

Figure 5.3-A1

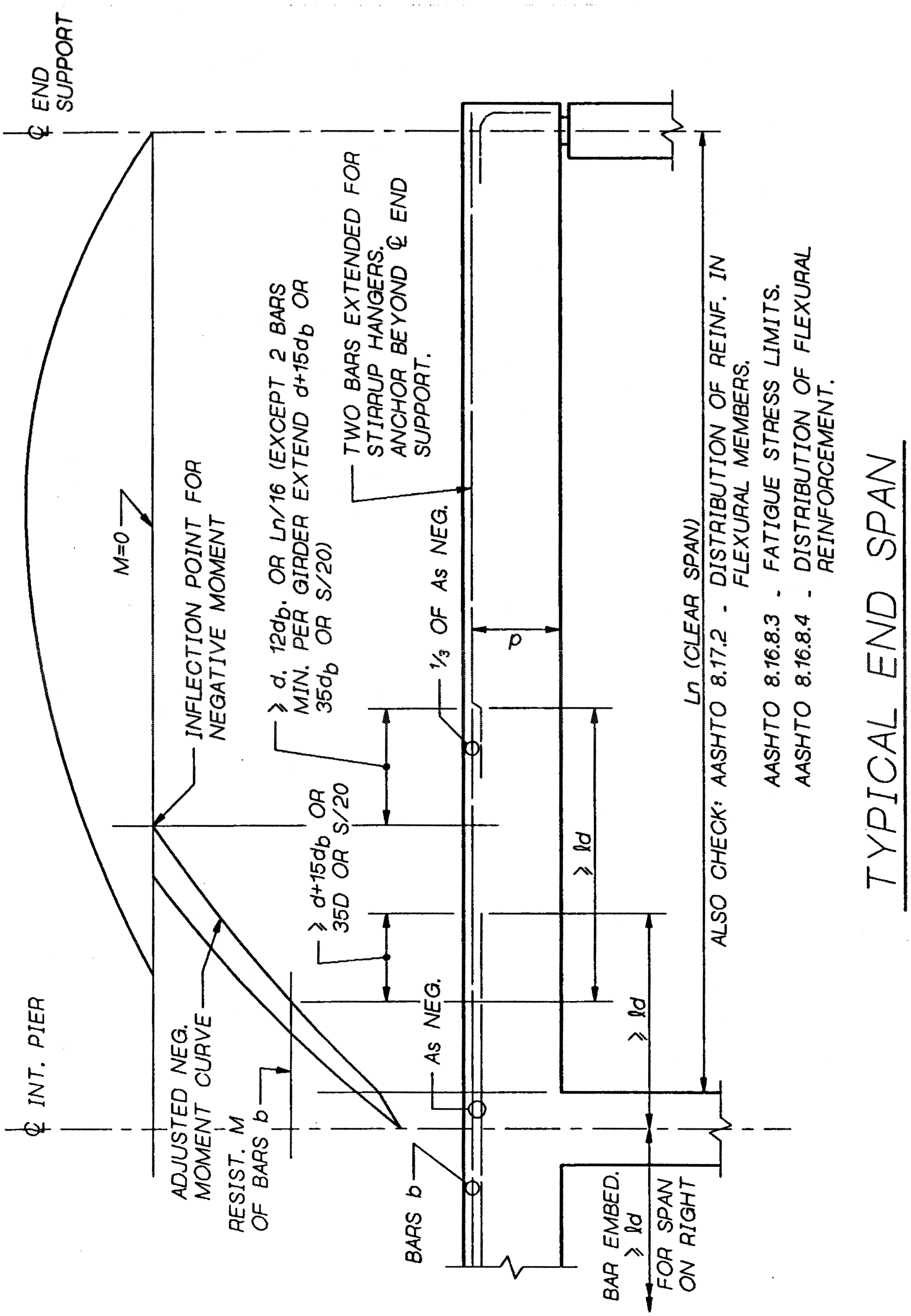
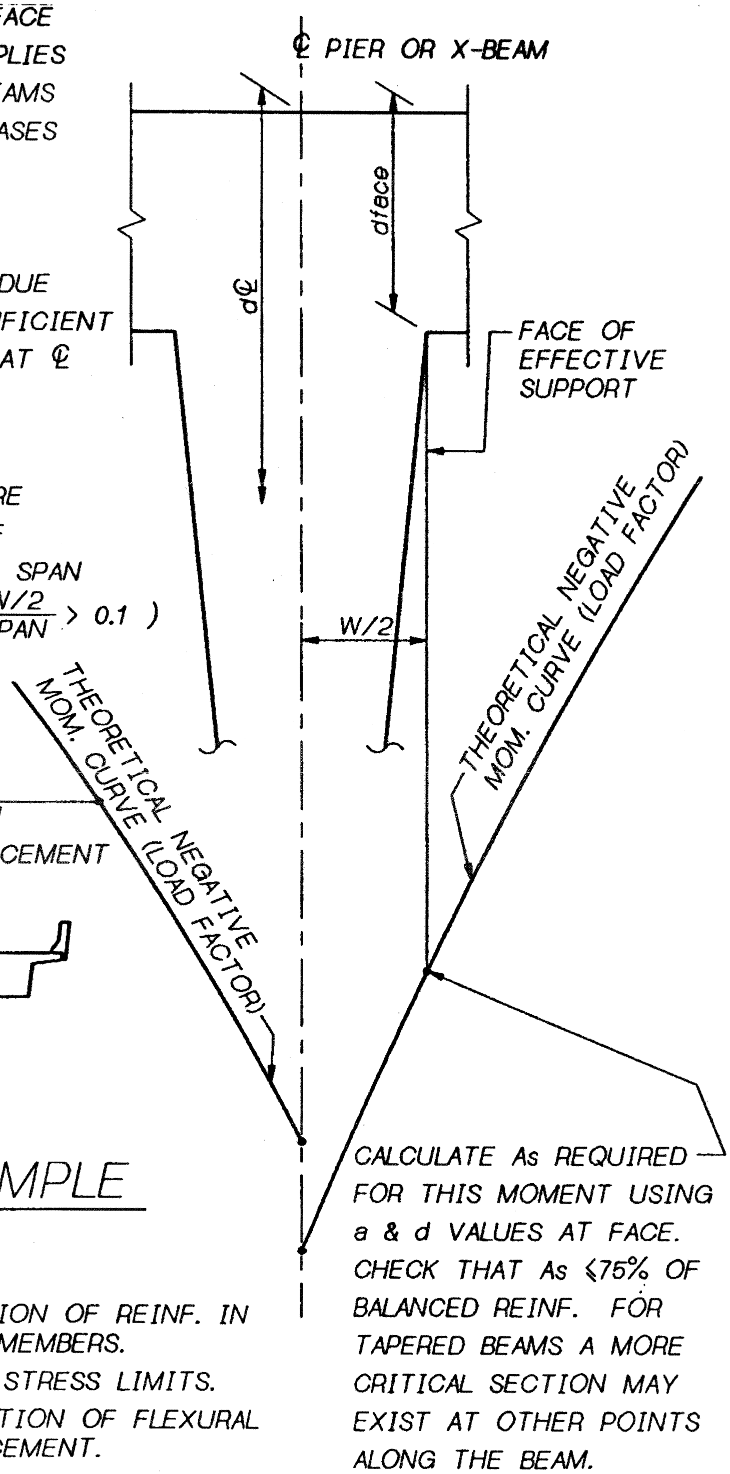

℄ END SUPPORT
℄ INT. PIER
M=0
INFLECTION POINT FOR NEGATIVE MOMENT
ADJUSTED NEG. MOMENT CURVE
RESIST. M OF BARS b
≥ d, 12db, OR Ln/16 (EXCEPT 2 BARS MIN. PER GIRDER EXTEND d+15db OR 35db OR S/20)
≥ d+15db OR 35D OR S/20
TWO BARS EXTENDED FOR STIRRUP HANGERS. ANCHOR BEYOND ℄ END SUPPORT.
1/3 OF As NEG.
As NEG.
BARS b
p
≥ ℓd
≥ ℓd
BAR EMBED. ≥ ℓd FOR SPAN ON RIGHT
Ln (CLEAR SPAN)
ALSO CHECK: AASHTO 8.17.2 - DISTRIBUTION OF REINF. IN FLEXURAL MEMBERS.
AASHTO 8.16.8.3 - FATIGUE STRESS LIMITS.
AASHTO 8.16.8.4 - DISTRIBUTION OF FLEXURAL REINFORCEMENT.
TYPICAL END SPAN

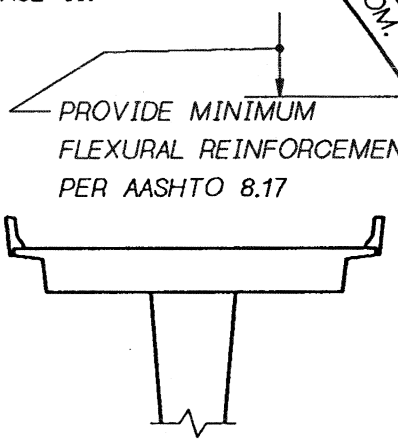
Figure 5.3-A2

CASE I (DESIGN FOR M AT FACE OF EFFECTIVE SUPPORT) APPLIES TO GIRDERS, BEAMS OR X-BEAMS WHERE THE SUPPORT INCREASES THE DEPTH OF THE BEAM <u>EXCEPT</u> FOR CASES WHERE:

1. THE INCREASE IN DEPTH DUE TO THE SUPPORT IS INSUFICIENT TO RESIST THE MOMENT AT ℄ SUPPORT; THAT IS $d℄ < d_{face} \frac{M\ 1/4\ PT.}{M_{face}}$
2. CONTINUOUS BEAMS WHERE ONE-HALF THE LENGTH OF SUPPORT DIVIDED BY THE SPAN IS GREATER THAN 0.1: $\left( \frac{W/2}{SPAN} > 0.1 \right)$

WHERE 1. OR 2. APPLIES USE CASE II.




## TYPICAL EXAMPLE

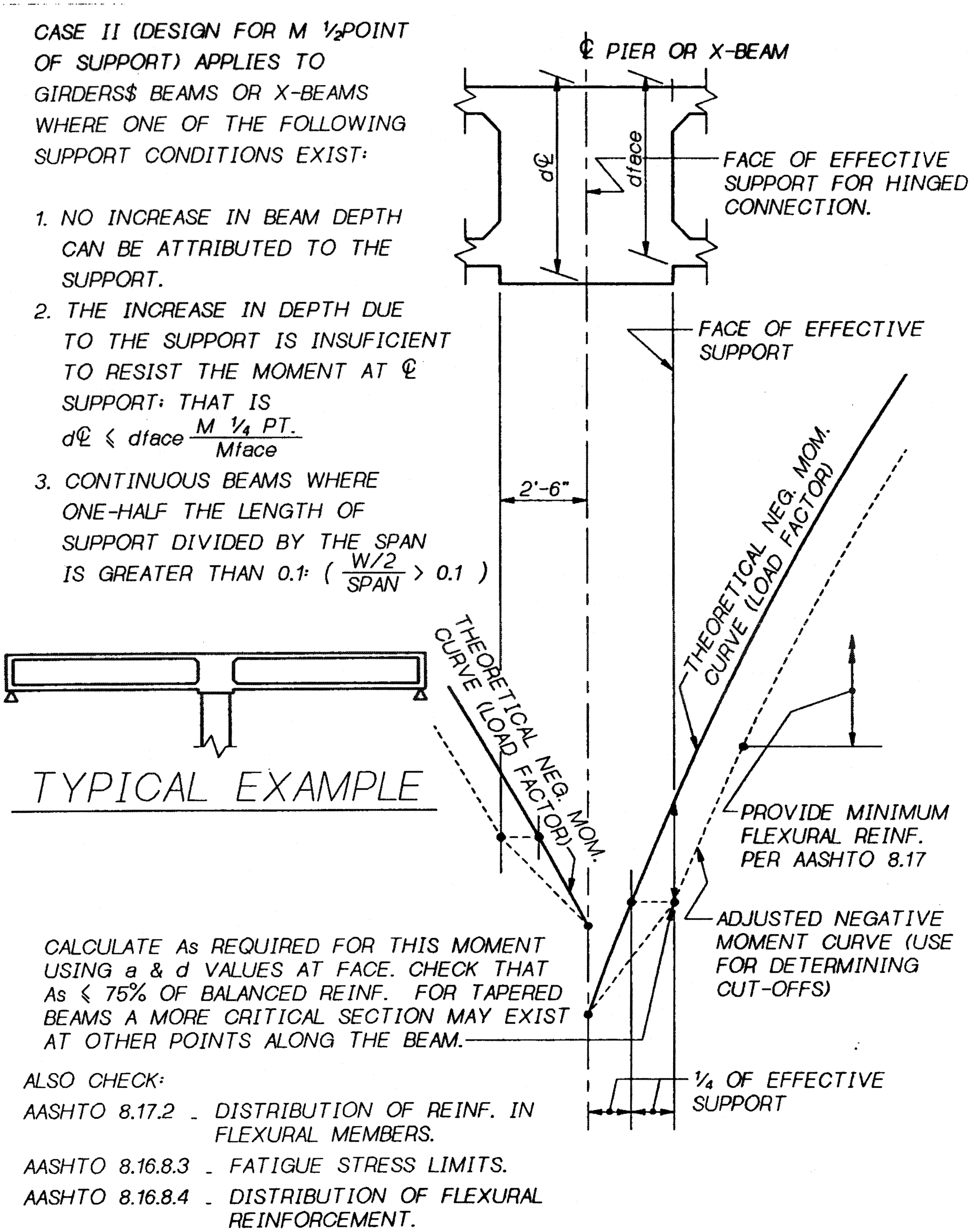
ALSO CHECK:

AASHTO 8.17.2 - DISTRIBUTION OF REINF. IN FLEXURAL MEMBERS.

AASHTO 8.16.8.3 - FATIGUE STRESS LIMITS.

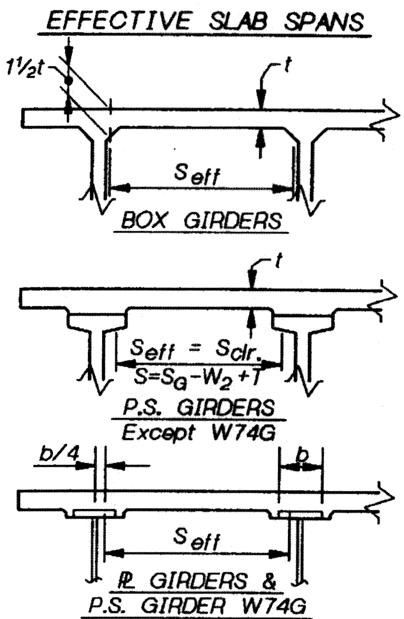
AASHTO 8.16.8.4 - DISTRIBUTION OF FLEXURAL REINFORCEMENT.

Figure 5.3-A3



CASE II (DESIGN FOR M ½POINT OF SUPPORT) APPLIES TO GIRDERS$ BEAMS OR X-BEAMS WHERE ONE OF THE FOLLOWING SUPPORT CONDITIONS EXIST:
1. NO INCREASE IN BEAM DEPTH CAN BE ATTRIBUTED TO THE SUPPORT.
2. THE INCREASE IN DEPTH DUE TO THE SUPPORT IS INSUFICIENT TO RESIST THE MOMENT AT ℄ SUPPORT: THAT IS
$d℄ \leqslant dface \frac{M\ 1/4\ PT.}{Mface}$
3. CONTINUOUS BEAMS WHERE ONE-HALF THE LENGTH OF SUPPORT DIVIDED BY THE SPAN IS GREATER THAN 0.1: ( $\frac{W/2}{SPAN} > 0.1$ )
TYPICAL EXAMPLE
℄ PIER OR X-BEAM
d℄
dface
FACE OF EFFECTIVE SUPPORT FOR HINGED CONNECTION.
FACE OF EFFECTIVE SUPPORT
2'-6"
THEORETICAL NEG. MOM. CURVE (LOAD FACTOR)
THEORETICAL NEG. MOM. CURVE (LOAD FACTOR)
PROVIDE MINIMUM FLEXURAL REINF. PER AASHTO 8.17
ADJUSTED NEGATIVE MOMENT CURVE (USE FOR DETERMINING CUT-OFFS)
CALCULATE As REQUIRED FOR THIS MOMENT USING a & d VALUES AT FACE. CHECK THAT As ⩽ 75% OF BALANCED REINF. FOR TAPERED BEAMS A MORE CRITICAL SECTION MAY EXIST AT OTHER POINTS ALONG THE BEAM.
¼ OF EFFECTIVE SUPPORT
ALSO CHECK:
AASHTO 8.17.2 _ DISTRIBUTION OF REINF. IN FLEXURAL MEMBERS.
AASHTO 8.16.8.3 _ FATIGUE STRESS LIMITS.
AASHTO 8.16.8.4 _ DISTRIBUTION OF FLEXURAL REINFORCEMENT.


Figure 5.3-A4




**GRADE 60 REINFORCING.**
$f'c = 4000$ psi

NOTES:

Reinforcing perpendicular to traffic.
$M_u = 1.3[DL + 5/3\ [LL + I]]$  $I = 30\%$
$M_{DL} = WS^2_{eff}/10$.  $M_{LL+I} = \frac{1}{32}[S_{eff} + 2][P_{25}][0.8][1.30]$
2½" clr. top, 1" clr. bottom for epoxy coated top mat of reinforcement.

When 2" clr. top is used (slab w/ overlay) decrease slab thickness called for on the chart by ½". The top ½" shall not be neglected when checking Distribution of Flexural Reinforcing Criteria.

Check cantilever overhang at exterior girders. Additional reinforcing may be required for traffic barrier load.

For continous spans only. Slab thickness shown is total thicknesss. Steel is for each surface of slab.

Min. $t = 12[[S_{eff} + 10]/30] \geq 7½"$ for epoxy coated top mat of rebars.

[Includes ½" wearing surface]. For post-tensioned box girders see AASHTO 9.9.1.

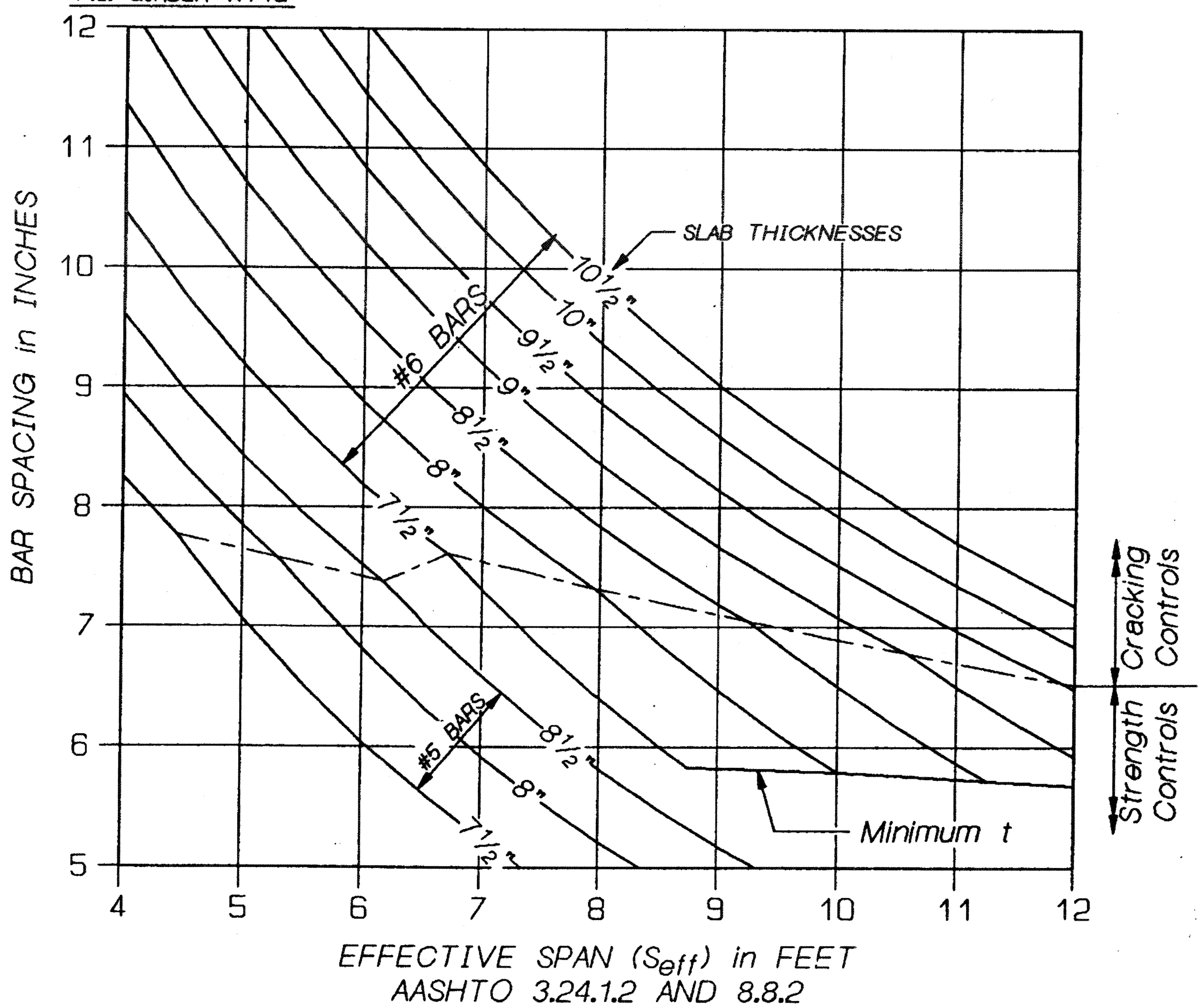
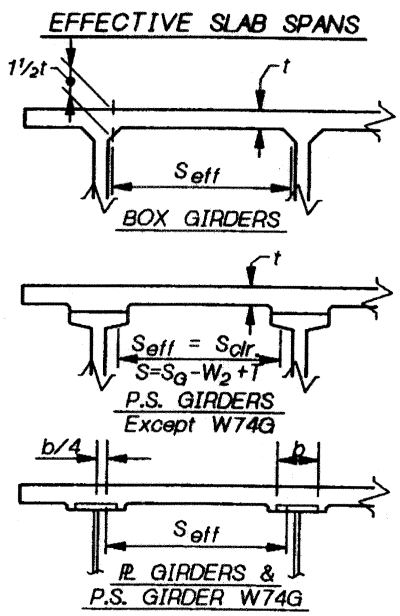



Figure 5.3-A5




GRADE 60 REINFORCING.
f'c = 5000 psi

NOTES:

Reinforcing perpendicular to traffic.
$M_u = 1.3[DL + 5/3\ [LL + I]]$ $I = 30\%$
$M_{DL} = WS^2_{eff}/10.$ $M_{LL+I} = \frac{1}{32}[S_{eff} + 2][P_{25}][0.8][1.30]$
2½" clr. top, 1" clr. bottom for epoxy coated top mat of reinforcement.

When 2" clr. top is used (slab w/ overlay) decrease slab thickness called for on the chart by ½". The top ½" shall not be neglected when checking Distribution of Flexural Reinforcing Criteria.

Check cantilever overhang at exterior girders. Additional reinforcing may be required for traffic barrier load.

For continous spans only. Slab thickness shown is total thicknesss. Steel is for each surface of slab.

Min. $t = 12[[S_{eff} + 10]/30] \geq 7\frac{1}{2}$" for epoxy coated top mat of rebars.

[Includes ½" wearing surface]. For post-tensioned box girders see AASHTO 9.9.1.

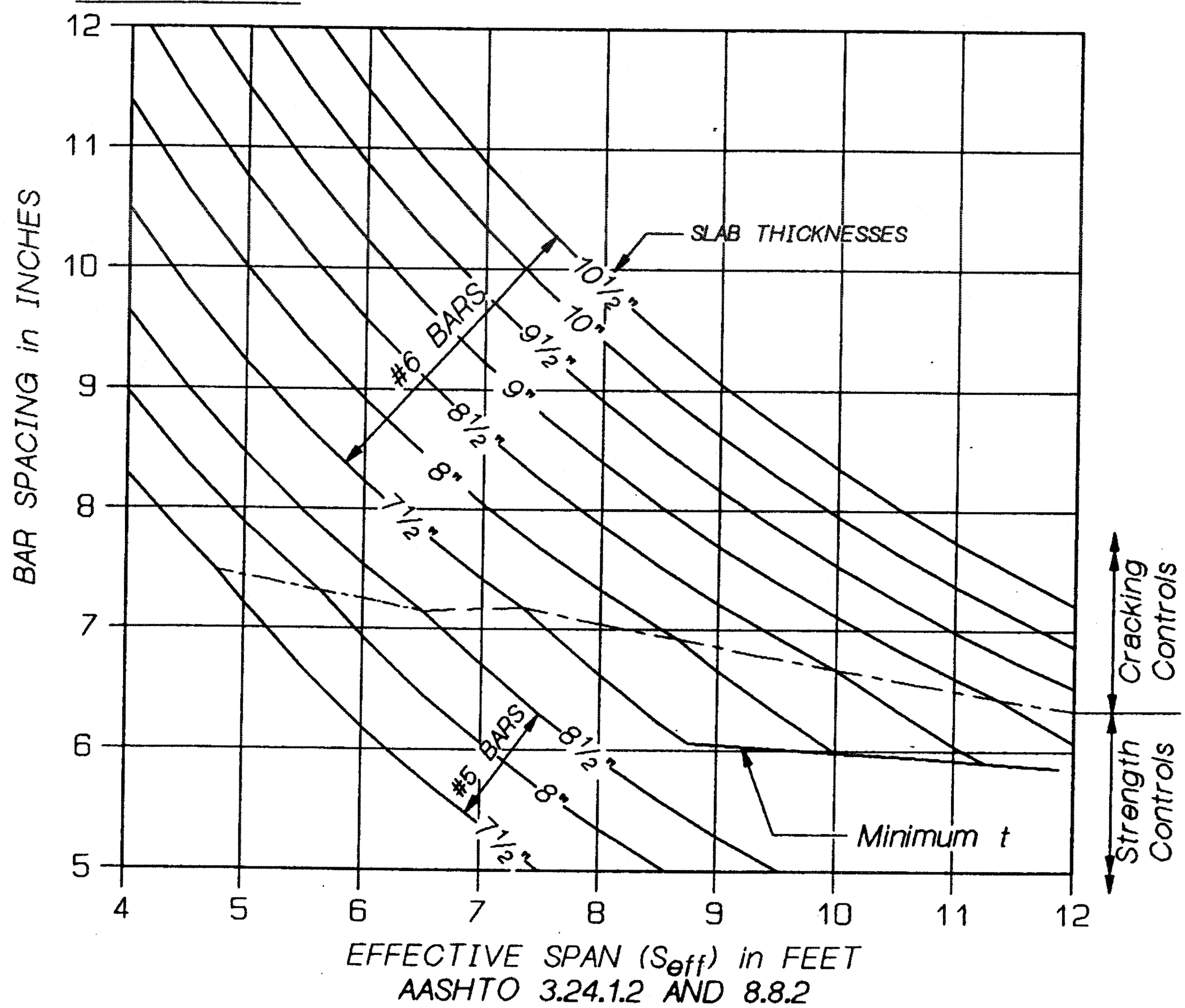
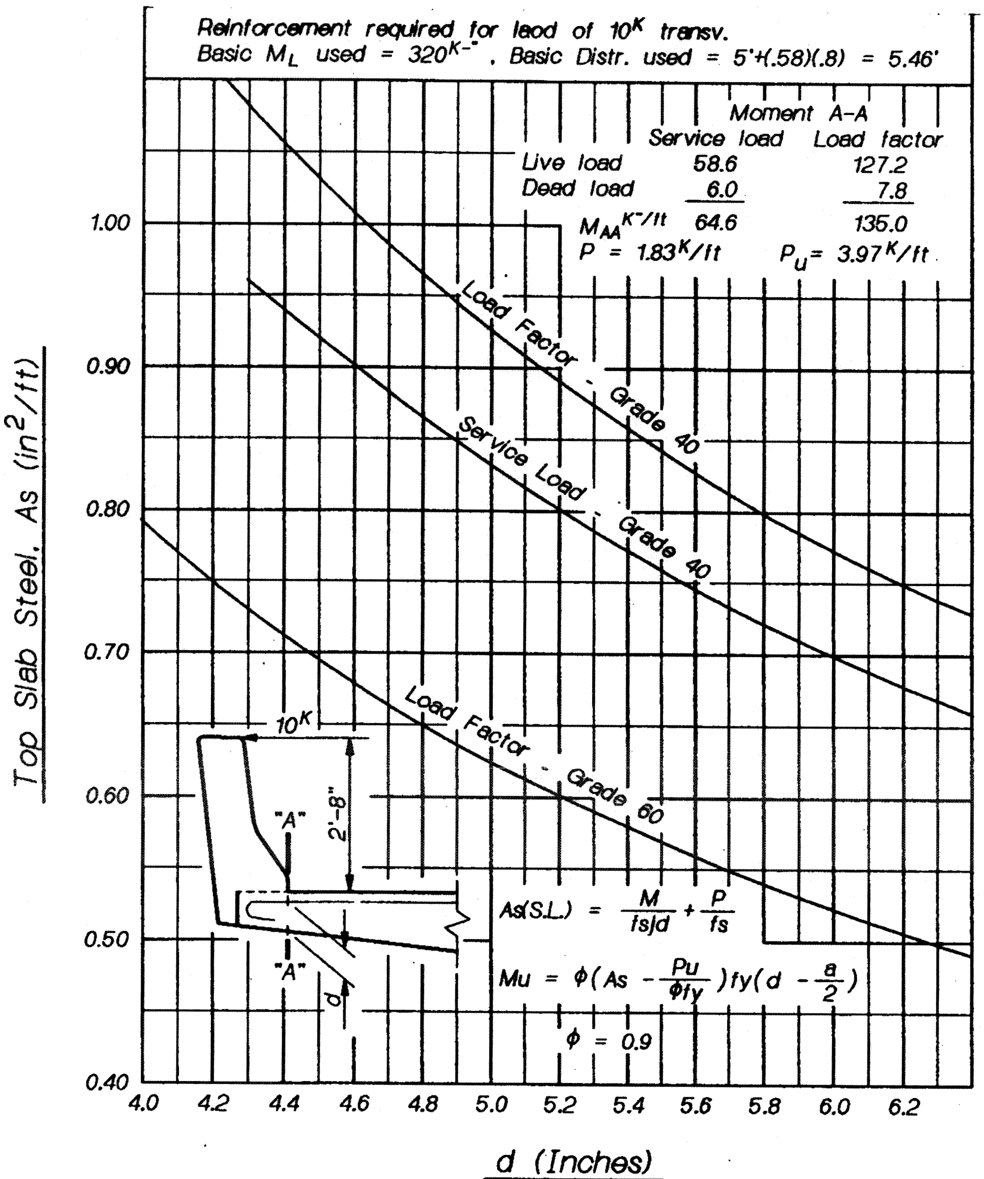



Figure 5.3-A6




*Notes:*

1. Section "A-A" is taken to be the critical section. Other sections ordinarily do not need to be investigated.
2. Provide enough extension to the left of "A-A" to develop the $A_s$ required (usually will require hooking bars).
3. Service Load $f_s$ = 20,000, Load Factor = (1.3D + 2.17L).
4. For Load Factor design, check distribution of flexural reinforcement — AASHTO 8.16-8.4. If #5 or #6 bars are used to furnish the $A_s$ from this chart, then this requirement will not have to be checked.

Figure 5.3-A7

## RECOMMENDATIONS FOR CONCRETE DECK SLAB DETAILING

These recommendations are primarily for beam-slab bridges with main reinforcement perpendicular to traffic.

- The minimum slab thickness including 0.5 inch wearing surface shall be 7.5 inches for concrete bridges and 8.0 inches for steel bridges.
- Minimum cover over the top layer of reinforcement shall be 2.5 inches including 0.5 inch wearing surface. Minimum cover over the bottom layer of reinforcements shall be 1.0 inch.
- Maximum bare size of #5 is preferred for all longitudinal and transverse reinforcements in deck slab except maximum bar size of #7 may be used for longitudinal reinforcements at intermediate piers.
- The minimum amounts of reinforcement shall be <u>0.18</u> in $^2$/ft of steel for top layer and <u>0.27</u> in $^2$/ft of steel for bottom layer. The maximum bar spacing shall be 15 inches for transverse and 18 inches for longitudinal reinforcements.
- Top and bottom reinforcement in both longitudinal and transverse directions of deck slab shall be staggered by at least 2 inches to allow better flowing of concrete and to minimize crack propagation into the slab.
- For bridges with skew angle less than 25 degrees, the primary reinforcement shall be placed parallel to the skew direction.
- For bridges with skew angle exceeding 25 degrees, the primary reinforcement shall be placed perpendicular to the girder direction.
- For bridges with skew angle exceeding 25 degrees, the amount of reinforcement in both primary and secondary directions **shall be increased** (up to double) in the end zone. Each end zone shall be taken as a longitudinal distance equal to the effective length of slab.
- A construction joint with roughened surface in the slab at the intermediate pier diaphragm shall be specified instead of construction joint with shear key.
- Both top and bottom layers of reinforcements at the intermediate pier shall be considered in negative moment reinforcement design.

## POST-TENSIONING SHOP DRAWING REVIEW

Post-tensioning shop drawings should be reviewed by the designer (or Bridge Technical Advisor for non bridge office projects) and consulted with the concrete specialist if needed. Shop drawings, after reviewed by the design engineer should be stamped with the official rubber seal and returned to the bridge construction support office.

**Review of the post-tensioning shop drawings should consist of the followings:**

**Post-tensioning Tendons:**

- All post-tensioning strands should be of 1/2" or 0.6" diameter grade 270 low relaxation uncoated strands. Tendon profile should be verified per contract plans. Duct size should be based on the duct area at least 2.5 times the total area of prestressing strands.
- Anchor set should conform to the contract plans. Anchor set of 1/4" is typically used for post-tensioned bridges.
- Maximum number of strands per tendon should not exceed 37 - 1/2" diameter strands or 27 - 0.6" diameter strands per Standard Specifications 6-02.3(26) D. Tendon Placement Patterns should be verified per contract plans.
- Jacking force per web should be verified per contract plans.
- Prestress force after anchor set (lift-off force) should conform to contract plans.
- Number of strands per web should be verified if specified in the contract plans. Anchorage system should conform to pre-approved list of post-tensioning system per BDM Appendix B. The anchorage assembly dimensions and reinforcement detailing should conform to the corresponding post-tensioning catalog.
- The curvature friction coefficient and wobble friction coefficient should conform to the contract plans. The curvature friction coefficient of p= 0.20 and a wobble friction coefficient of k = 0.0002 k/ft are often used. These coefficients may be revised by the post-tensioning supplier if approved by the design engineer and conform to the Standard Specifications 6.02.3(26) E.
- Post-tensioning stressing sequence should be in accordance with the contract plan post-tensioning Notes.
- Tendon stresses shall not exceed as specified per BDM figure 6.4.2-2:
  a. 0.79fpu at anchor ends immediately before seating.
  b. 0 70fpu at anchor ends immediately after seating.
  c. 0.75fpu at the end point of length influenced by anchor set.
- Elongation calculations for each jacking operation should be verified. If the difference in tendon elongation exceeds 2%, the elongation calculations should be separated for each tendon per Standard Specification 6-02.3(26) A.
- Vent points should be provided at all high points along tendon.
- Drain holes should be provided at all low points along tendon.

**Concrete Strength:**

- The concrete strength at the time of post-tensioning, f'ci should be less than 4000 psi per standard specifications 6-02.3(26) E-1. Different concrete strength may be used if specified in the contract plans.
- Concrete stresses at the anchorage should be checked per Standard specifications 6-02.3(26) B-1 for bearing type anchorage. For other type of anchorage assemblies, if not covered in the BDM Appendix B for pre-approved list of posttensioning system, testing per Standard Specifications 6-02.3(26) B-2 is required.

**During Construction**

**Acceptance:**

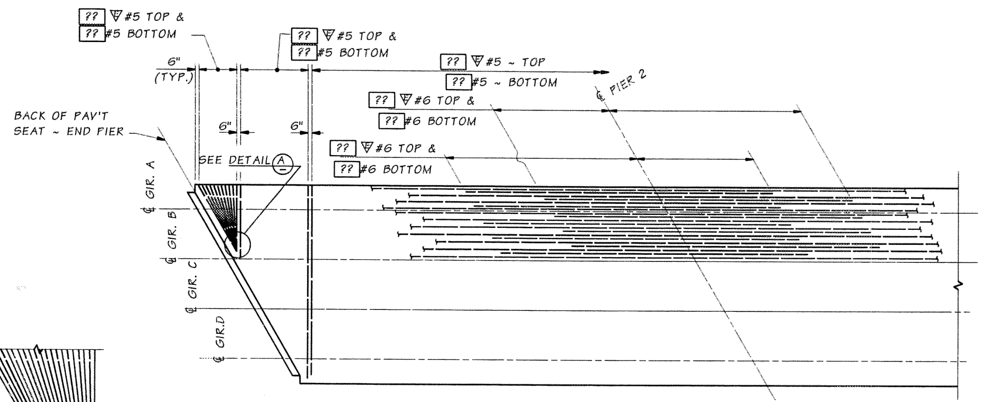
- If the measured elongation of each strand tendon is within +/- 7% of the approved calculated elongation, the stressed tendon is acceptable.
- If the measured elongation is greater than 7%, force verification after seating (lift off force) is required. The lift-off force should not be less than 99% of the approved calculated force or more 70% fpu As.
- If the measured elongation is less than 7%, the bridge construction office will instruct the force verification.

**Problems and recommended solutions.**

- Broken Strands.
  One broken strand per tendon is usually acceptable. (Post-tensioning design should preferably allow one broken strand). If more than one strand per tendon is broken, the group of tendon per we should be considered. If a group of tendon in a web is under stressed per design, the design engineer should investigate the adequacy of the entire structure.
- Over or under elongation
  This is usually taken care by the bridge construction office.
- Low concrete strength The design engineer should investigate the adequacy of design with lower strength.
- Failed anchorage
  This is usually taken care by the bridge construction office.
- Other problems such as unbalanced and out of sequence post-tensioning, delayed post-tensioning due to mechanical problems, etc. should be evaluated per case-by-case basis.

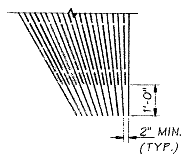
**Post-Tensioning Notes**

1. The cast-in-place concrete in superstructure shall be Class???. The minimum compressive strength of the cast-in-place concrete at the time of post-tensioning shall be ??? ksi.
2. The minimum prestressing load after seating for each web shall be ??? kips. Each web shall have a minimum of ?? strands.
3. The design is based on ?? inch (mm) diameter low relaxation strands with a jacking load of ??? kips each web, an anchor set of 3/8 inch, a curvature friction coefficient, = 0.20 and a wobble friction coefficient, k = 0.0002 k/ft (0.00066 KN/m). The actual anchor set used by the contractor shall be specified in the shop plans and included in the transfer force calculations.
4. The design is based on the estimated prestress loss of post-tensioned prestressing strands of ??? ksi (MPa) due to steel relaxation, elastic shortening, creep and shrinkage of concrete.
5. The contractor shall submit the stressing sequence and elongation calculations to the engineer for approval. All losses due to tendon vertical and horizontal curvature must be included in elongation calculations. The stressing sequence shall meet the following criteria:
   - A. The prestressing force shall be distributed with an approximately equal amount in each web and shall be placed symmetrically about the centerline of the bridge.
   - B. No more than one-half of the prestressing force in any web may be stressed before an equal force is stressed in the adjacent webs. At no time during stressing operation will more than one-sixth of the total prestressing force be applied eccentrically about the centerline of bridge.
6. The maximum outside diam/eter of the duct shall be ??? inches. The area of the duct shall be at least 2.5 times the net area of the prestressing steel in the duct.
7. All tendons shall be stressed from pier ?? .

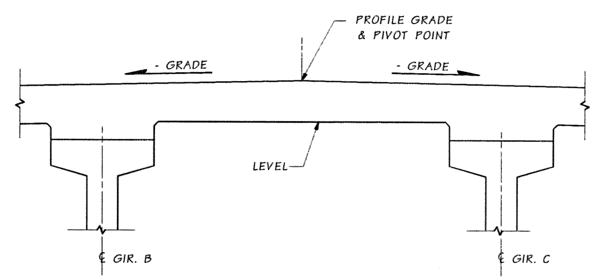



ROADWAY SLAB REINFORCEMENT

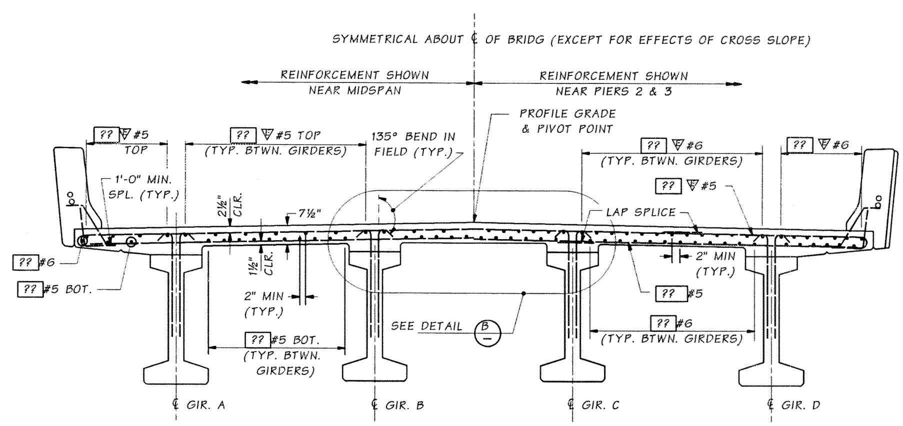
FOR SKEW ANGLE > 25°, TRANSVERSE REINFORCEMENT SHALL BE PLACE IN THE DIRECTION OF THE SKEW



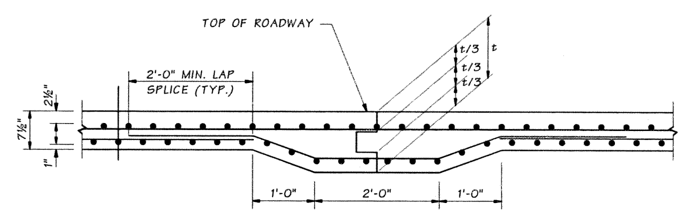

DETAIL A




DETAIL B




TYPICAL SECTION REINFORCEMENT




ALTERNATIVE TRANSVERSE CONSTRUCTION JOINT IN SLAB

t = TOP OF SLAB TO TOP OF GIRDER (1'-0" MAX.)

| | | | | | |
|---|---|---|---|---|---|
| Bridge Design Engr. | M:\BRIDGELIB\Master6\Standards\Girders\Wide Flange\TYPICAL SLAB PLAN_SECTION .man | | | | |
| Supervisor | REGION NO. | STATE | FED. AID PROJ. NO. | SHEET NO. | TOTAL SHEETS |
| Designed By | 10 | WASH. | | | |
| Checked By | JOB NUMBER | | | | |
| Detailed By | | | | | |
| Bridge Projects Engr. | | | | | |
| Prelim. Plan By | | | | | |
| Architect/Specialist | DATE | REVISION | BY | APP'D | |

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD PRESTRESSED CONCRETE GIRDERS

SLAB REINFORCEMENT PLAN AND TYPICAL SECTION

BRIDGE SHEET NO.

SHEET OF SHEETS

SR JOB NO. SHEET

**Example 5.2-B1**

Given: Center-to-center spacing of girders = 12 feet 3 inches
Width of top flange of steel girder = 18 inches wide
Deck concrete, Class 4000 $f_c'$ = 4,000 psi
Reinforcing steel, Grade 60 $f_y$ = 60,000 psi
Cover to top bars = 2.5 inches
Cover to bottom bars = 1.0 inch
Analyze a 1 foot wide section of slab

Find: Deck thickness, deck reinforcement

**1. Determine Deck Thickness**

$S_{eff} = 12.25' - 2\,(18'')\,/\,(4)\,(12) = 11.50'$

Minimum thickness, $t_{min} = (S_{eff} + 10)\,(12)\,/\,30 = (11.50 + 10)\,(12)\,/\,30 = 8.60''$

Use 8¾″ thick slab

**2. Determine Transverse Deck Reinforcement — Top Slab Reinforcement**

Dead Load Moment, $M_{DL}$:

$M_{DL} = (1/10)\,[\,(8.75''\,/\,12)\,(0.160\text{ kcf})\,]\,(11.50)^2 = 1.55$ kip-ft/ft

Live Load Moment + Impact, $M_{LL+I}$:

$$M_{LL+I} = \frac{(S+2)}{32}\,(P_{wheel})\,(0.8)\,(1.30)$$ AASHTO, 1989, Section 3.24.3.1

where: $P_{wheel} = 1.25$ (16 kips/wheel) = 20.0 kips/wheel (HS25 Truck)

continuity factory = 0.8 — AASHTO, 1989, Section 3.24.3.1
impact factor = 1.30

$$M_{LL+I} = \frac{(11.50+2)}{32}\,(20.0)\,(0.8)\,(1.30) = 8.78 \text{ kip-ft/ft}$$

Factored Design Moment, $M_u$:

$M_u = 1.3\,[\,1.55 + (5/3)\,(8.78)\,] = 21.04$ kip-ft/ft

Determine $A_{s\ req'd}$: $d_{top\ bars} = 8.75 - 2.5 - (0.75)\,/\,2 = 5.875''$

$M_u\,/\,(\phi)\,(b)\,(d)^2 = 21.04\,(12{,}000)\,/\,(0.9)\,(12)\,(5.875)^2 = 677.3$ psi

Interpolating from Table 5.2-A2, Appendix A: $\rho = 0.01272$

$A_{s\ req'd} = \rho\,(b)\,(d) = 0.01272\,(12)\,(5.875) = 0.90\ \text{in}^2/\text{ft}$

**Use #6 bars at 5″ ctrs, $A_s$ = 1.06 in$^2$/ft > 0.90 in$^2$/ft ok**

Use same bar size and spacing for bottom slab reinforcement. An alternate approach is to solve directly for $A_{s\ req'd}$ from Eq (5), BDM Section 5.2.1B:

$$A_{s\ req'd} = 0.85\,(f_c'\,/\,f_y)\,(b)\,[\,d - \sqrt{d^2 - (31.3725\,M_u\,/\,f_c'\,b)}\,] \quad (5)$$

$$= [\,0.85\,(4)\,(12)\,/\,60\,]\,[\,5.875 - \sqrt{(5.785)^2 - 31.3725\,(21.04)\,/\,(4)\,(12)}\,]$$

$A_{s\ req'd} = 0.90\ \text{in}^2/\text{ft}$ Agrees with value previously computed by tables.

Check $A_{s\,min}$ using Table 5.2-A2, Appendix A:

$M_u = 1.2\,M_{cr} = 1.2\,f_r\,S = (1.2)\ 7.5\sqrt{f_c'}\ (1/6)\,(b)\,(t)^2$

$= (1.2)\ 7.5\sqrt{4{,}000}\ \ (1/6)\,(12)\,(8.75)^2\ \ 87{,}160$ in-lbs/ft

$M_u / \phi\,bd^2 = 87{,}160 / [\,0.9\,(12)\,(5.875)^2\,] = 233.8$ psi

From Table 5.2-A2, Appendix A, interpolate $\rho = 0.00404$

$A_{s\,min} = \rho\,(b)\,(d) = 0.00404\,(12)\,(5.875) = 0.28\ in^2/ft < 1.06\ in^2/ft$

Check $A_{s\,min}$ using Eq (6):

$$A_{s\,min} = \frac{0.85\,f_c'\,(b)}{f_y}\left(d - \sqrt{d^2 - \frac{0.124\,h^2}{\sqrt{f_c'}}}\right) \qquad (6)$$

$$A_{s\,min} = \frac{0.85\,(4)\,(12)}{(60)}\left(5.875 - \sqrt{(5.875)^2 - \frac{0.124\,(8.75)^2}{\sqrt{4}}}\right)$$

$A_{s\,min} = 0.285\ in^2/ft$ Agrees with value from tables.

Check $A_{s\,max}$: From Table 5.2-A2, Appendix A, $\rho_{max} = 0.75\,\rho_b = 0.0214$

$A_{s\,max} = 0.0214\,(12)\,(5.875) = 1.51\ in^2/ft$

Check $A_{s\,max}$ using Eq (7), BDM Section 5.2.1B:

$$A_{s\,max} = 0.6375\,\beta_1\,(b)\,(d)\,\frac{f_c'}{f_y}\left(\frac{87}{87 + f_y}\right) \qquad (7)$$

$$A_{s\,max} = 0.6375\,(0.85)\,(12)\,(5.875)\,\frac{(4)}{(60)}\left(\frac{87}{87 + 60}\right) = 1.51\ in^2/ft \qquad \text{ok}$$

**3. Check Crack Control Requirements**

Calculate $f_s$ due to Service Load:

$M_{service\ load} = 1.55 + 8.78 = 10.33$ kip-ft/ft

$f_{s\,calc} = M(12{,}000) / A_s jd$

Where $j = 1 - k/3 = 0.884$ Agrees with Table 1, page 81, ACI Publication SP-3 *Reinforced Concrete Design Handbook Working Stress Design*, 1965

$k = 1 / 1\,[\,1 + f_s/nf_c] = 1 / [\,1 + 24{,}000 / (8)\,(1{,}600)\,] = 0.348$

$f_s = 24{,}000$ psi Grade 60 bars per AASHTO, Section 8.15.2.2

$f_c = 0.40\,f_c' = 1{,}600$ psi for Conc Cl 4000

$n = E_s / E_c = 29{,}000{,}000 / 3{,}620{,}000 = 8.0$

$f_{s\,calc} = 10.33\,(12{,}000) / (1.06)\,(0.884)\,(5.875) = 22{,}517$ psi

Using Eq (21), BDM Section 5.2.1G, Calculate allowable $f_s$:

$$f_{s\,allowable} = z / [\,(d_c)\,(A)\,]^{1/3} \qquad \text{Eq (21)}$$

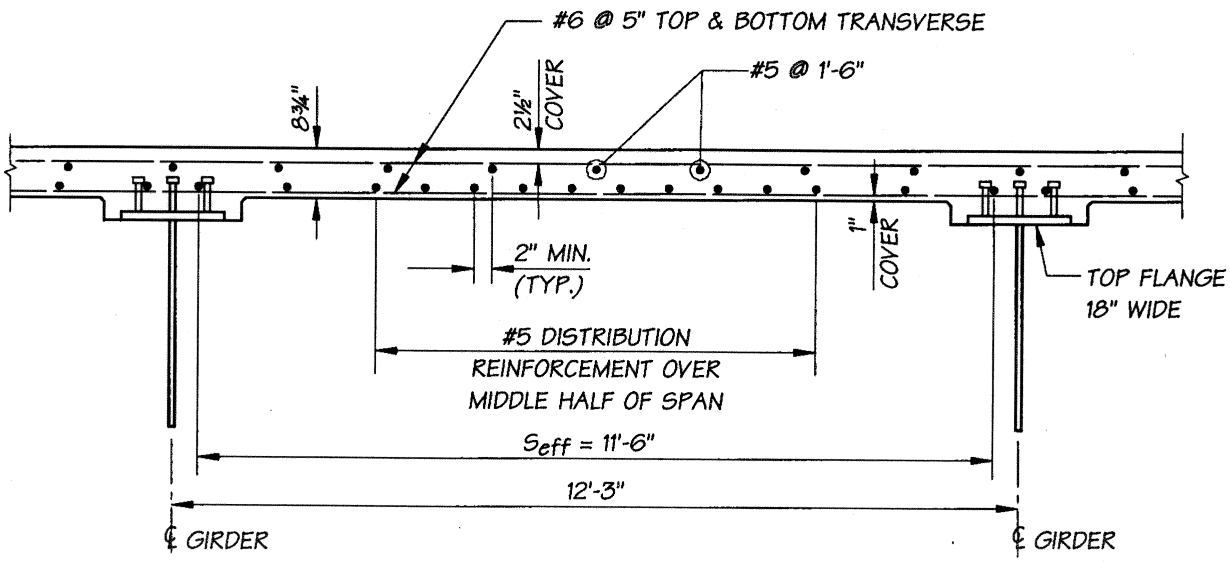
$$= 130 / [\,(2.875)\,(5)\,(5.75)\,]^{1/3} = 29.63\ ksi > 22.52\ ksi \qquad \text{ok}$$

Alternate Approach, Check $z_{calc} < 130$ kips/in using Eq (22):

$$z_{calc} = f_{s\,calc} \, [ \, (d_c) \, (A) \, ]^{1/3} < 130 \text{ kips/in} \qquad \text{Eq (22)}$$

$$= (22.52) \, [ \, (2.875) \, (5) \, (5.75) \, ]^{1/3} = 98.1 \text{ kips/in} < 130 \text{ kips/in} \qquad \text{ok}$$

**Use #6 bars at 5″ ctrs top and bottom transverse slab reinforcement.**




**Deck Reinforcement — Mid-Span Steel Plate Girder**

**Example 5.2-B2**

Given: Center-to-center spacing of W58G girders = 8 feet 0 inches
Width of top flange = 25 inches wide
Average flange thickness = 6 inches
Girder concrete strength $f_c'$ = 7,000 psi
Deck concrete, Class 5000 $f_c'$ = 5,000 psi
Cover to top bars = 2.5 inches
Cover to bottom bars = 1.0 inch

Find: Deck thickness, deck reinforcement

**1. Determine Deck Thickness**

Minimum slab thickness = 7.5″ no overlay, per BDM, Chapter 6. This thickness permits the use of #6 transverse and #5 longitudinal bars.

$S_{eff}$ = clear span per AASHTO 3.24.1.2(a)

Width of top flange/average flange thick = 4.16

Close enough to 4.0, use clear span for $S_{eff}$

$S_{eff} = S_g - W_2 = 8.0' - 2.083' = 5.92'$

Check Minimum Slab Thickness, $t_{min}$:

$t_{min} = (S_{eff} + 10)(12) / 30 = (5.92' + 10)(12) / 30 = 6.37'' < 7.5''$ ok

**2. Determine Transverse Deck Reinforcement — Top Slab Reinforcement**

Dead Load Moment, $M_{DL}$:

$M_{DL} = (1/10)\,[\,(7.5'' / 12)(0.160\text{ kcf})\,]\,(5.92)^2 = 0.43$ kip-ft/ft

Live Load Moment + Impact, $M_{LL+I}$:

$$M_{LL+I} = \frac{(S+2)}{32}(P_{wheel})(0.8)(1.30) = \frac{(6.54+2)}{32}(20.0)(0.8)(1.30)$$

$M_{LL+I} = 5.15$ kip-ft/ft

Factored Design Moment, $M_u$:

$M_u = 1.3\,[\,0.35 + (5/3)(5.15)\,] = 11.61$ kip-ft/ft

Determine $A_{s\ req'd}$: $d_{top\ bars} = 7.5 - 2.5 - (0.75)/2 = 4.625''$

$M_u / (\phi)(b)(d)^2 = 12.54\,(12{,}000) / (0.9)(12)(4.625)^2 = 651.4$ psi

Interpolating from Table 5.2-A3, Appendix A: $\rho = 0.01089$

$A_{s\ req'd} = \rho\,(b)(d) = 0.01089\,(12)(4.625) = 0.61\ \text{in}^2/\text{ft}$

**Use #6 bars at 8″ ctrs, $A_s$ = 0.66 in$^2$/ft ok**

Use same bar size and spacing for bottom slab reinforcement.

**3. Check Crack Control Requirements — Transverse Reinforcement**

Calculate $f_s$ due to Service Load:

$M_{service\ load} = 0.35 + 5.15 = 5.50$ kip-ft/ft

$f_{s\ calc} = M\,(12{,}000) / A_s jd$

where:
$j = 1 - k/3 = 1 - 0.375/3 = 0.875$
$k = 1/1\,[\,1 + f_s/nf_c\,] = 1/[\,1 + 24{,}000/(7.2)\,(2{,}000)\,] = 0.375$
$f_c = 0.40\,f_c' = (0.40)\,(5{,}000) = 2{,}000$ psi for Concrete Class 5000
$E_c = 57{,}000\ \sqrt{5{,}000} = 4{,}030{,}500$ psi
$f_s = 24{,}000$ psi Grade 60 bars
$n = E_s / E_c = 29{,}000{,}000 / 4{,}030{,}500 = 7.2$

$f_{s\ calc} = 5.50\,(12{,}000) / (0.66)\,(0.875)\,(4.625) = 24{,}710$ psi top bar

Calculate $f_{s\ allowable} = z / (Ad_c)^{1/3}$:

$A = (7.5'')\,(2.875'')\,(2) / 1$ bar $= 43.125$ $\quad$ $dc = 2.5 + 0.75/2 = 2.875''$

$f_{s\ allow} = 130 / [\,(43.125)\,(2.875)\,]^{1/3} = 26.07$ ksi $> 24.71$ ksi $\quad$ ok

**4. Determine Longitudinal Deck Reinforcement**

Moments at Pier, Negative Reinforcement:

$M_{DL} = 187.6$ kip-ft/girder $\quad$ $M_{LL+I} = 780.0$ kip-ft/girder $\quad$ Service Load Moments

$M_u = 1.3\,[\,187.6 + (5/3)\,(780.0)\,] = 1{,}933.8$ kip-ft/girder

Determine $A_{s\ req'd}$ assume two layers of #5 with $d_{avg} = 64.0''$:

$M_u / (\phi)\,(b)\,(d)^2 = 1{,}933.8\,(12{,}000) / (0.9)\,(25)\,(64)^2 = 251.8$ psi

Interpolating from Table 5.2-A3, Appendix A: $\rho = 0.00433$

$A_{s\ req'd} = 0.00433\,(25)\,(64.0) = 6.93\ in^2$

**Use 24-#5 (12-#5 in each layer) $A_s = 7.44\ in^2 > 6.93\ in^2$ $\quad$ ok**

Spacing is approximately 8.0″, $A_s/ft = 0.47\ in^2/ft$

Check longitudinal distribution reinforcement so that spacing can be coordinated with the reinforcement required for negative pier girder moment:

$P = 220/\sqrt{S} = 220/\sqrt{6.54} = 86.0$ percent but not to exceed 67 percent

Distribution Reinforcement $= 0.67\,(A_{s\ actual}) = 0.67\,(0.70) = 0.47\ in^2/ft$

$A_{s\ provided} = 0.47\ in^2/ft$ $\quad$ ok

**5. Check Crack Control Requirement — Longitudinal Reinforcement**

24-#5 $A_s = 7.44 \text{ in}^2$ $n = E_s/E_c = 29{,}000{,}000 / 4{,}769{,}000 = 6.0$

$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$

$k = \sqrt{2(0.0047)(6.0) + [(0.0047)(6.0)]^2} - (0.0047)(6.0) = 0.210$

$j = 1 - k/3 = 0.93$

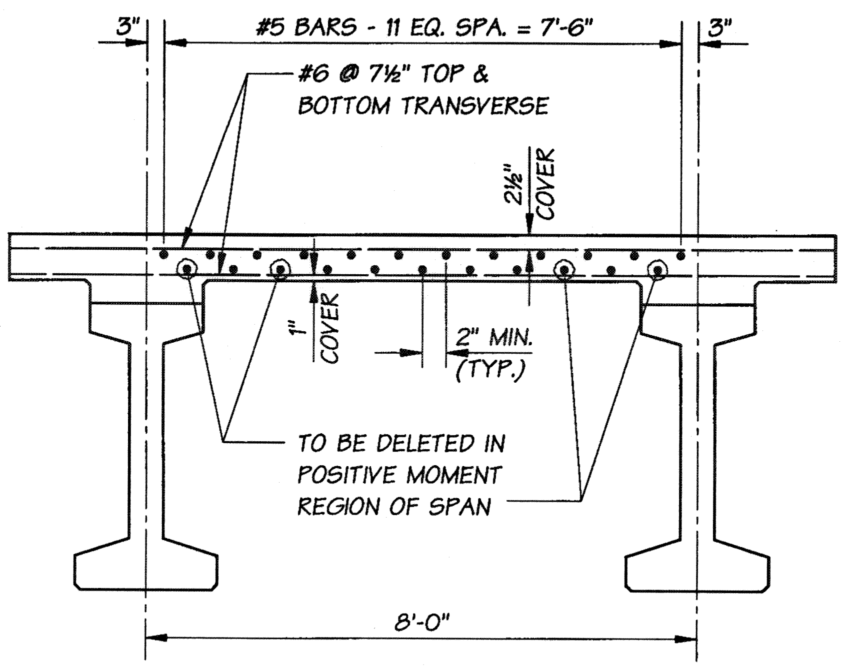
$f_{s\ calc} = M(12{,}000) / A_s jd = 967.6(12{,}000) / (7.44)(0.93)(64.0) = 26{,}220$ psi

$f_{s\ allowable} = z / [(d_c)(A)]^{1/3}$

Use actual girder spacing = (8.0′) (12) = 96.0″ to compute A

$A = (96)(7.5) / 24$ bars $= 30.0 \text{ in}^2$/bar   $d_c = 2.5 + 0.75 + 0.625/2 = 3.56''$

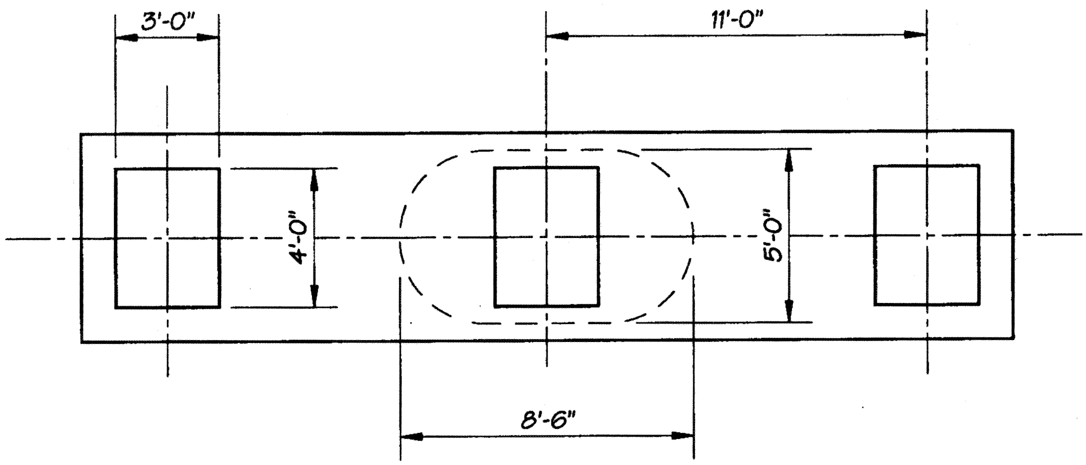
$f_{s\ allowable} = 130 / [30.0(3.56)]^{1/3} = 27.40$ psi $> 26.22$ psi   ok



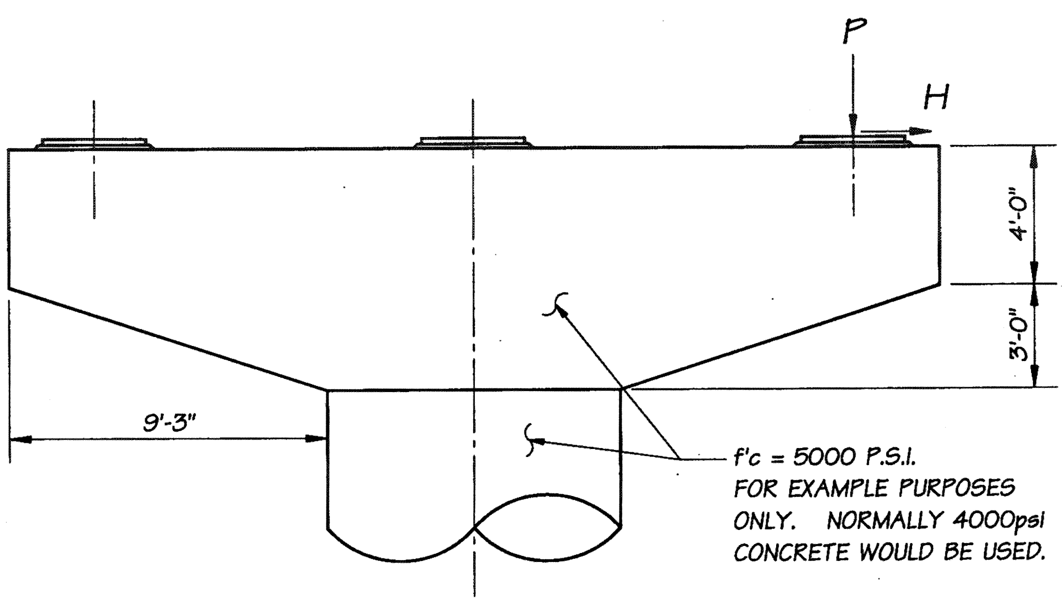

**Deck Reinforcement at Intermediate Pier — Prestressed Girder Bridge**

Longitutdinal Deck Reinforcement is designed for the negative moment at an intermediate pier. Otherwise, the longitudinal deck reinforcement will be similar to that shown in Example 5.2-B1-1.

**Example 5.2-B3**




PLAN - PIER CAP

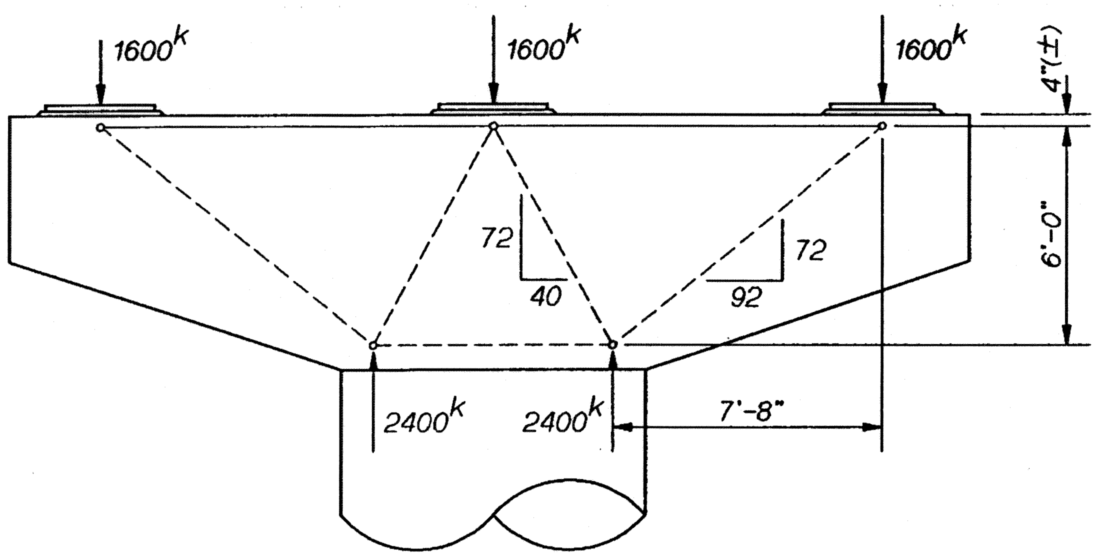



ELEVATION - PIER CAP

**Design Loads**

Group I: $P_u = 1600^k$ $H = 0$
Group VII: $P_u = 1500^k$ $H = 400^k$
Assume crossbeam dead load is included with bearing loads.
Use Section 12.4 of AASHTO's *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 1989.

**Develop a Preliminary Strut-and-Tie Model:**




STRUT & TIE DIAGRAM

Estimate node size at top of column:

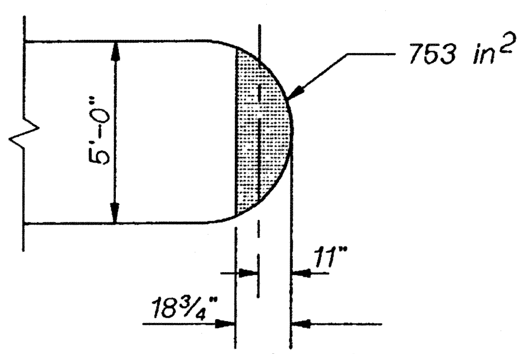
$\phi_b\ (f_{cn}\ A_{cn}) \ \geq\ S_u$

Assuming spiral reinforcement provides confinement, use $\phi_b\ =\ 0.75$ and $f_{cn}\ =\ 0.85\ f_c'$:

$0.75\ (0.85\ \times\ 5)\ A_{cn}\ \geq\ 2{,}400$

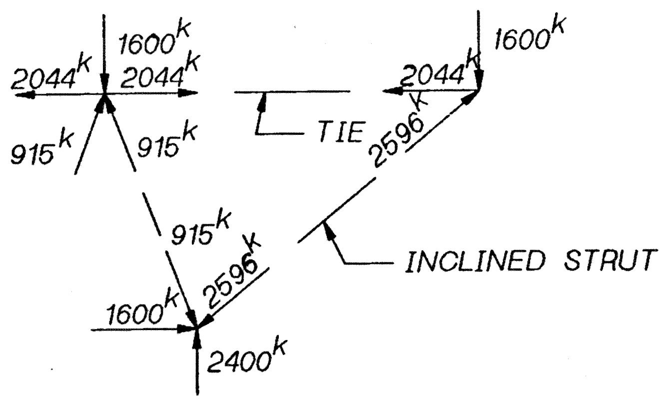
$A_{cn}\ \geq\ 753\ \text{in}^2$

Use the following node size at the top of column:

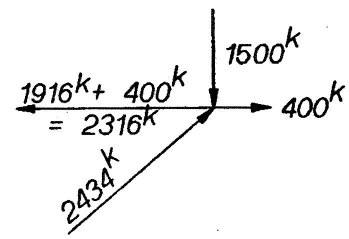



NODE SIZE AT TOP OF COLUMN

**Determine Truss Element Forces:**




**Group I Strut Loads**




**Group VII Strut Loads**

**Determine Minimum Size of Node Regions:**

$\phi_b\,(f_{cn}\,A_{cn}) \geq S_u$ where: $\phi_b = 0.70$ for bearing

$f_{cn} = 0.85\,f_c'$ in regions with compression only

$f_{cn} = 0.70\,f_c'$ in regions with one tension tie

At base of inclined strut,

$$0.75\,(0.85 \times 5)\;A_{cn} \geq 2{,}596$$

$$A_{cn} \geq 873\ \text{in}^2$$

$$\text{depth of node} = \frac{873}{72''} = 12.1'' \qquad (72'' \times 12.1'')$$

where width of crossbeam = 72″

At top of inclined strut, $A_{cn} \geq \dfrac{2{,}596}{0.70\,(0.70 \times 5)} = 1{,}060\ \text{in}^2$

$$\text{depth of node} = \frac{1{,}060}{72''} = 14.7'' \qquad (72'' \times 14.7'')$$

For $1{,}600^k$ chord: $A_{cn} \geq \dfrac{1{,}600}{0.70\,(0.85 \times 5)} = 538\ \text{in}^2$

$$\text{depth of node} = \frac{538}{72''} = 7.5''$$

For $915^k$ chord: $A_{cn} \geq \dfrac{915}{1{,}600}\,(538) = 308\ \text{in}^2$

$$\text{depth of node} = \frac{308}{72''} = 4.3''$$

**Determine Minimum Sizes of Compression Members:**

$\phi_v\,(f_{cu}\,A_{cs}) \geq S_u$ (inclined compressive struts)

$\phi_f\,(0.85\,f_c'\,A_{cc} + A_s'\,f_s') \geq S_u$ (compression chords)

For 2,596$^k$ inclined compressive strut:

$0.85\,(0.45 \times 5)\;A_{cs} \geq 2{,}596^k$ ($f_{cu} = 0.45\,f_c'$)

$$A_{cs} \geq \frac{2{,}596}{0.85\,(0.45)\,(5)} = 1{,}357\ \text{in}^2$$

$$\text{and depth of strut} = \frac{1{,}357}{72} = 18.9\ \text{in}$$

For 915$^k$ inclined compressive strut:

$$A_{cs} \geq \frac{915}{2{,}596}\,(1{,}357) = 478\ \text{in}^2$$

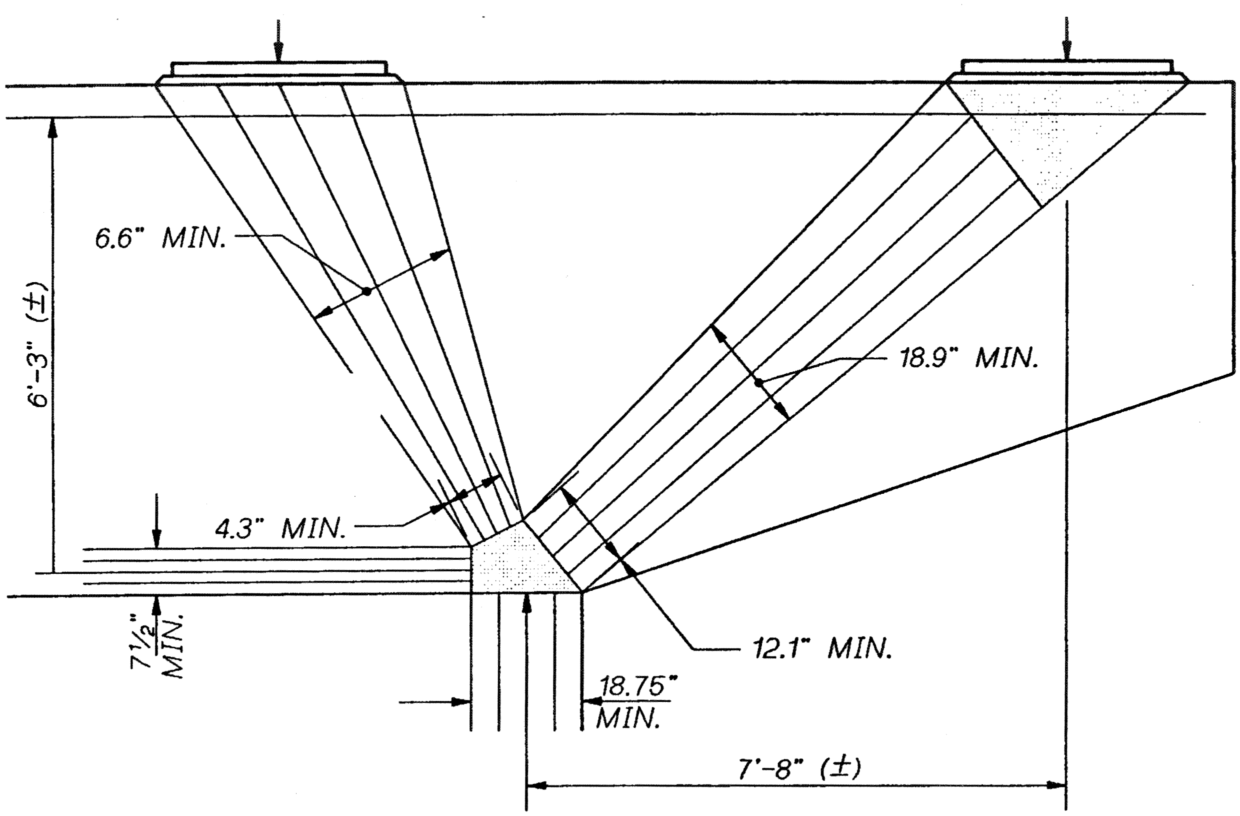
$$\text{and depth of strut} = \frac{478}{72} = 6.6\ \text{in}$$

For 1,600$^k$ compression chord:

$$A_{cs} \geq \frac{1{,}600}{0.9\,(0.85)\,(5)} = 418\ \text{in}^2$$

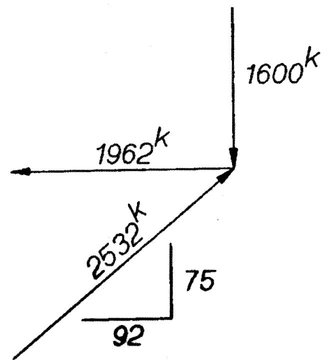
$$\text{and depth of chord} = \frac{418}{72} = 5.8\ \text{in}$$

**Incorporate Node and Member Sizes Into Model:**

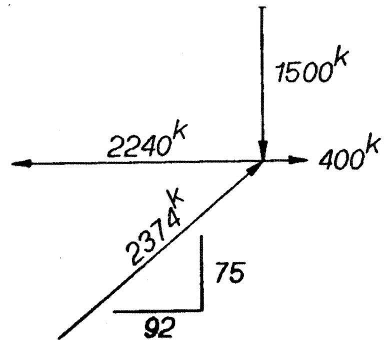



STRUT & NODE GEOMETRY

**Recalculate Truss Member Forces:**




**Group I Strut Loads**




**Group VII Strut Loads**

**Design Tie Member:**

$\phi_f (A_s f_{sy} + A*_s f*_{su}) \geq S_u$

without prestress: $0.90 (A_s)(60) \geq 2{,}240$

$A_s \geq 41.5 \text{ in}^2$

Try using 12 bundles of #14 top and #11 bot ($A_s = 45.7 \text{ in}^2$)

Check development length of tie bars:

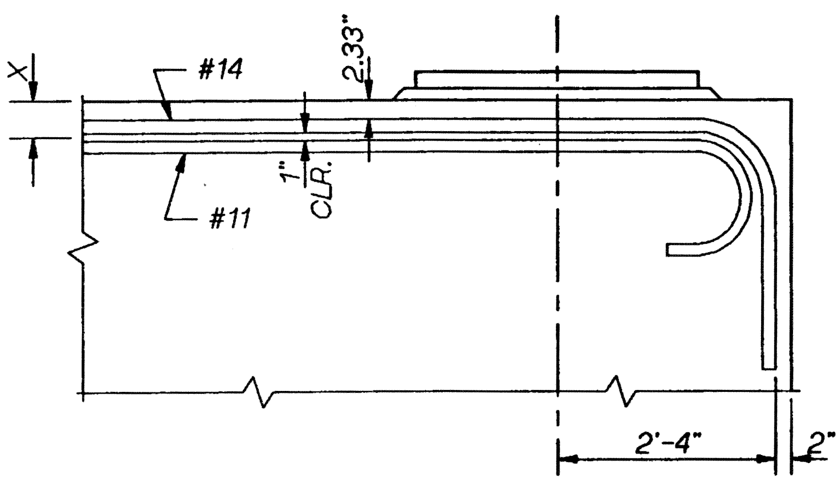
For #14 bars with $f_c' = 5{,}000$ psi, $l_{dh} = 2' - 5''$

Development length available $= 2' - 4'' < 2' - 5''$

For #11 bars, $l_{dh} = 1' - 5''$ ok

Therefore, total developed steel $A_s = 12\,(1.56) + 12\,(2.25)\left(\frac{28}{29}\right)$

$A_s = 44.8 \text{ in}^2 > 41.5 \text{ in}^2$ ok




Partial Elevation-Tension Tie at Top of Pier Cap

$$x = \frac{12\,(2.25)\,(3.26) + 12\,(1.56)\,(5.97)}{45.7} = 4.37'' = 4'' \text{ estimate} \quad \text{ok}$$

**Determine Minimum Vertical and Horizontal Steel Using Sections 12.5.3.2 and 12.5.3.3:**

For vertical reinforcing: $A_s f_y \geq 120\ b_w\ s$

where $s < \frac{d}{4}$ or 12″

Therefore, $A_s \geq \frac{120\ b_w\ s}{60{,}000} = 0.002\ b_w\ s$

Assume 4 legs of #6 stirrups: $A_s = 1.76\ \text{in}^2$

$$s \leq \frac{A_s}{0.002\ b_w} = \frac{1.76}{0.002\ (72)}$$

$$s \leq 12.2\ \text{in}$$

Check: $\frac{d}{4} = \frac{72 - 4.37}{4} = 16.9''$

Therefore, use 4 #6 legs at 12″ maximum spacing.

For horizontal reinforcing: $A_s f_y \geq 120\ b_w\ s$

where $s < \frac{d}{3}$ or 12″

For $s = 12''$, $A_s \geq 0.002\ (72)\ (12) = 1.73\ \text{in}^2$ (2 – #9 bars)

Try 2 #8 bars: $A_s = 1.58\ \text{in}^2$

$$s \leq \frac{1.58}{0.002\ (72)} = 11.0''$$

Use #8 bars at 11″ maximum spacing on side faces.

For bottom bars, use #6 at approximately 12″ (7 – #6 bars)

## Example 5.2-B4

### Service Load — Concrete Stresses and Constants

| | | CLASS | CLASS |
|---|---|---|---|
| $n$ | (See $E_c$ below) | 8 | 10 |
| $f'_c$ | | 4000 psi. | 3000 psi. |
| $f_c$ | (Compression) | 1600 | 1200 |
| $f_t$ | (Tension) Use only with special permission | 100 | 86 |
| $f_s$ | (Grade 40) | 20,000 | 20,000 |
| $f_s$ | (Grade 60) | 24,000 | 24,000 |
| $v_c$ | (With web reinf.) | 313 | 271 |
| $v_c$ | | 60 * | 52 * |
| | Slabs & Footings (Peripheral Shear) | | |
| | $v_c$ | 114 | 99 |
| | $v_c$ (With web reinf.) | 190 | 164 |
| $K$ | ($f_s = 24000$) | 246 | |
| $K$ | Balanced rectangular sections | 272 | 197 |
| $k$ | Balanced rectangular sections | .390 | .375 |
| $j$ | Balanced rectangular sections | .870 | .875 |
| $p$ | Balanced rectangular sections | .0156 | .01125 |
| $E_c$ | (for stress calc.) ($n$ as above) | 522,000 K/☐" | 418,000 K/☐" |
| $E_c$ | (for short term defl. due to E. Q., etc.) ($n = 8$) | 522,000 | 522,000 |
| $E_c$ | (for D.L. Camber of Slabs, T&S Moms., Settlement) ($n = 16$) | 261,000 | 261,000 |
| $E_c$ | (for D.L. Camber, except slabs) ($n = 24$) | 174,000 | 174,000 |

Temp. Coeff. = .000006 %/° ~ 45° Drop to 35° Rise ~ All climates.
Shrinkage Coeff. = .0002 % (Temp. rise & shrinkage cancel).
* For more detailed analysis $v_c = 0.9(f'_c)^{\frac{1}{2}} + 1100\, p_w\left(\frac{Vd}{M}\right)$
See 1974 A.A.S.H.T.O. Interim 1.5.29 (B)(2).

Stirrup spacing; (Kip & inch units)

$$S = \frac{A_s \times f_s \times jd}{V - V_c} = \frac{A_s \times 20 \times 7/8\, d}{V_s} = \frac{17.50\, A_s \times d}{V_s}$$

$A_s$ = Total area of stirrup legs.
$V_s$ = Total shear taken by stirrups.
$V$ = Total shear on section.
$V_c$ = Total shear by conc. = $v_c \times bjd$

o

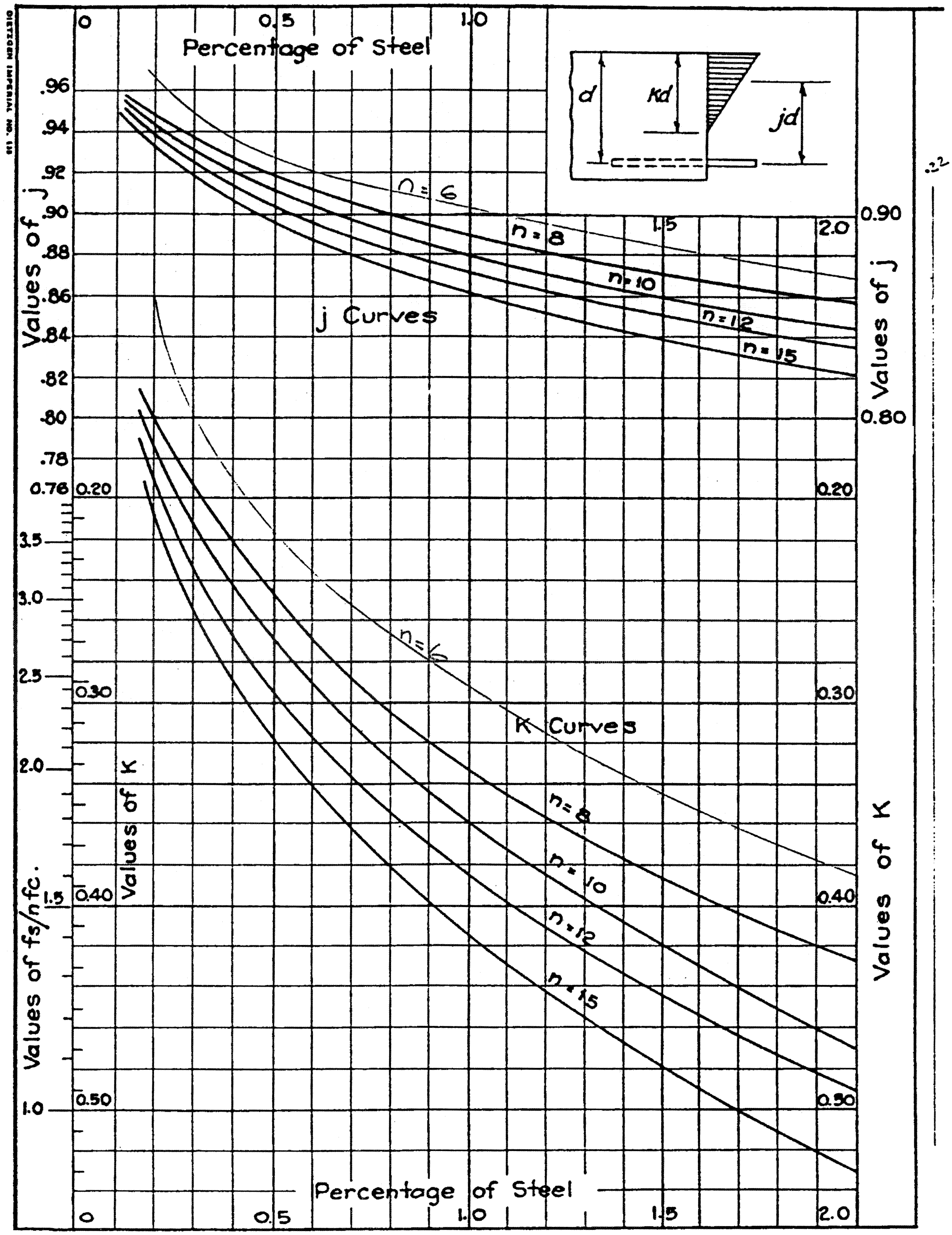
$$d = \sqrt{M/bK} \quad \text{(Balanced rectangular section)}$$

$$f_c = \frac{2M}{kjbd^2} \quad \text{(Rectangular section)}$$

$$f_s = \frac{M}{A_s jd}$$

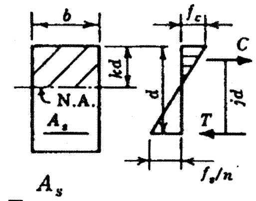
$$v = \frac{V}{bjd}$$

$$n = \frac{E_s}{E_c}$$


Percentage of Steel
0
0.5
1.0
1.5
2.0
d
Kd
jd
j Curves
n = 6
n = 8
n = 10
n = 12
n = 15
K Curves
Values of j
Values of K
Values of fs/nfc.
.96
.94
.92
.90
.88
.86
.84
.82
.80
.78
0.76
0.90
0.80
0.20
0.30
0.40
0.50
3.5
3.0
2.5
2.0
1.5
1.0

## COEFFICIENTS ($K$, $k$, $j$, $p$) FOR RECTANGULAR SECTIONS

| $f'_c$ and $n$ | $f_c$ | $K$ | $k$ | $j$ | $p$ | $K$ | $k$ | $j$ | $p$ |
|---|---|---|---|---|---|---|---|---|---|
| | | $f_s$ = 16,000 | | $a$ = 1.13 | | $f_s$ = 18,000 | | $a$ = 1.29 | |
| | 875. | 137. | .356 | .881 | .0097 | 128. | .329 | .890 | .0080 |
| 2500 | 1000. | 169. | .387 | .871 | .0121 | 158. | .359 | .880 | .0100 |
| | 1125. | 201. | .415 | .862 | .0146 | 190. | .387 | .871 | .0121 |
| 10.1 | 1250. | 235. | .441 | .853 | .0172 | 222. | .412 | .863 | .0143 |
| | 1500. | 306. | .486 | .838 | .0228 | 291. | .457 | .848 | .0190 |
| | 1050. | 173. | .376 | .875 | .0124 | 162. | .349 | .884 | .0102 |
| 3000 | 1200. | 212. | .408 | .864 | .0153 | 199. | .380 | .873 | .0127 |
| | 1350. | 252. | .437 | .854 | .0184 | 238. | .408 | .864 | .0153 |
| 9.2 | 1500. | 294. | .463 | .846 | .0217 | 278. | .434 | .855 | .0181 |
| | 1800. | 380. | .509 | .830 | .0286 | 362. | .479 | .840 | .0240 |
| | 1400. | 249. | .412 | .863 | .0180 | 234. | .384 | .872 | .0149 |
| 4000 | 1600. | 303. | .444 | .852 | .0222 | 286. | .416 | .861 | .0185 |
| | 1800. | 359. | .474 | .842 | .0266 | 341. | .444 | .852 | .0222 |
| 8.0 | 2000. | 417. | .500 | .833 | .0313 | 397. | .471 | .843 | .0261 |
| | 2400. | 536. | .545 | .818 | .0409 | 513. | .516 | .828 | .0344 |
| | 1750. | 327. | .437 | .854 | .0239 | 309. | .408 | .864 | .0199 |
| 5000 | 2000. | 397. | .470 | .843 | .0294 | 376. | .441 | .853 | .0245 |
| | 2250. | 468. | .500 | .833 | .0351 | 446. | .470 | .843 | .0294 |
| 7.1 | 2500. | 542. | .526 | .825 | .0411 | 518. | .497 | .835 | .0345 |
| | 3000. | 694. | .571 | .810 | .0535 | 666. | .542 | .819 | .0452 |




$$p = \frac{A_s}{bd}$$

$$k = \frac{1}{1 + f_s/nf_c} \qquad j = 1 - \tfrac{1}{3}k$$

$$p^* = \frac{f_c}{2f_s} \times k \qquad K = \frac{f_c}{2}kj$$

$$a = \frac{f_s}{12{,}000} \times (\text{av. } j\text{-value})$$

for use in

$$A_s = \frac{M}{ad} \text{ or } A_s = \frac{NE}{adi}$$

| $f'_c$ and $n$ | $f_c$ | $K$ | $k$ | $j$ | $p$ | $K$ | $k$ | $j$ | $p$ | $K$ | $k$ | $j$ | $p$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $f_s$ = 20,000 | | $a$ = 1.44 | | $f_s$ = 22,000 | | $a$ = 1.60 | | $f_s$ = 24,000 | | $a$ = 1.76 | |
| | 875. | 120. | .306 | .898 | .0067 | 113. | .287 | .904 | .0057 | 107. | .269 | .910 | .0049 |
| 2500 | 1000. | 149. | .336 | .888 | .0084 | 141. | .315 | .895 | .0072 | 133. | .296 | .901 | .0062 |
| | 1125. | 179. | .362 | .879 | .0102 | 170. | .341 | .886 | .0087 | 161. | .321 | .893 | .0075 |
| 10.1 | 1250. | 211. | .387 | .871 | .0121 | 200. | .365 | .878 | .0104 | 191. | .345 | .885 | .0090 |
| | 1500. | 277. | .431 | .856 | .0162 | 264. | .408 | .864 | .0139 | 253. | .387 | .871 | .0121 |
| | 1050. | 152. | .326 | .891 | .0085 | 144. | .305 | .898 | .0073 | 136. | .287 | .904 | .0063 |
| 3000 | 1200. | 188. | .356 | .881 | .0107 | 178. | .334 | .889 | .0091 | 169. | .315 | .895 | .0079 |
| | 1350. | 226. | .383 | .872 | .0129 | 214. | .361 | .880 | .0111 | 204. | .341 | .886 | .0096 |
| 9.2 | 1500. | 265. | .408 | .864 | .0153 | 252. | .385 | .872 | .0131 | 240. | .365 | .878 | .0114 |
| | 1800. | 346. | .453 | .849 | .0204 | 331. | .429 | .857 | .0176 | 317. | .408 | .864 | .0153 |
| | 1400. | 221. | .359 | .880 | .0126 | 210. | .337 | .888 | .0107 | 199. | .318 | .894 | .0093 |
| 4000 | 1600. | 272. | .390 | .870 | .0156 | 258. | .368 | .877 | .0134 | 246. | .348 | .884 | .0116 |
| | 1800. | 324. | .419 | .860 | .0188 | 309. | .396 | .868 | .0162 | 295. | .375 | .875 | .0141 |
| 8.0 | 2000. | 379. | .444 | .852 | .0222 | 362. | .421 | .860 | .0191 | 347. | .400 | .867 | .0167 |
| | 2400. | 492. | .490 | .837 | .0294 | 472. | .466 | .845 | .0254 | 454. | .444 | .852 | .0222 |
| | 1750. | 292. | .383 | .872 | .0168 | 278. | .361 | .880 | .0144 | 265. | .341 | .886 | .0124 |
| 5000 | 2000. | 358. | .415 | .862 | .0208 | 341. | .392 | .869 | .0178 | 326. | .372 | .876 | .0155 |
| | 2250. | 426. | .444 | .852 | .0250 | 407. | .421 | .860 | .0215 | 390. | .400 | .867 | .0187 |
| 7.1 | 2500. | 496. | .470 | .843 | .0294 | 475. | .447 | .851 | .0254 | 456. | .425 | .858 | .0221 |
| | 3000. | 641. | .516 | .828 | .0387 | 617. | .492 | .836 | .0335 | 595. | .470 | .843 | .0294 |
| | | $f_s$ = 27,000 | | $a$ = 2.00 | | $f_s$ = 30,000 | | $a$ = 2.24 | | $f_s$ = 33,000 | | $a$ = 2.48 | |
| | 875. | 99. | .247 | .918 | .0040 | 92. | .228 | .924 | .0033 | 86. | .211 | .930 | .0028 |
| 2500 | 1000. | 124. | .272 | .909 | .0050 | 115. | .252 | .916 | .0042 | 108. | .234 | .922 | .0036 |
| | 1125. | 150. | .296 | .901 | .0062 | 140. | .275 | .908 | .0052 | 132. | .256 | .915 | .0044 |
| 10.1 | 1250. | 178. | .319 | .894 | .0074 | 167. | .296 | .901 | .0062 | 157. | .277 | .908 | .0052 |
| | 1500. | 237. | .359 | .880 | .0100 | 224. | .336 | .888 | .0084 | 211. | .315 | .895 | .0072 |
| | 1050. | 126. | .264 | .912 | .0051 | 117. | .244 | .919 | .0043 | 110. | .226 | .925 | .0036 |
| 3000 | 1200. | 157. | .290 | .903 | .0064 | 147. | .269 | .910 | .0054 | 138. | .251 | .916 | .0046 |
| | 1350. | 190. | .315 | .895 | .0079 | 178. | .293 | .902 | .0066 | 168. | .273 | .909 | .0056 |
| 9.2 | 1500. | 225. | .338 | .887 | .0094 | 211. | .315 | .895 | .0079 | 199. | .295 | .902 | .0067 |
| | 1800. | 299. | .380 | .873 | .0127 | 282. | .356 | .881 | .0107 | 267. | .334 | .889 | .0091 |
| | 1400. | 185. | .293 | .902 | .0076 | 173. | .272 | .909 | .0063 | 162. | .253 | .916 | .0054 |
| 4000 | 1600. | 230. | .322 | .893 | .0095 | 215. | .299 | .900 | .0080 | 203. | .279 | .907 | .0068 |
| | 1800. | 277. | .348 | .884 | .0116 | 260. | .324 | .892 | .0097 | 246. | .304 | .899 | .0083 |
| 8.0 | 2000. | 326. | .372 | .876 | .0138 | 308. | .348 | .884 | .0116 | 291. | .327 | .891 | .0099 |
| | 2400. | 430. | .416 | .861 | .0185 | 407. | .390 | .870 | .0156 | 387. | .368 | .877 | .0134 |
| | 1750. | 247. | .315 | .895 | .0102 | 231. | .293 | .902 | .0085 | 218. | .274 | .909 | .0073 |
| 5000 | 2000. | 305. | .345 | .885 | .0128 | 287. | .321 | .893 | .0107 | 271. | .301 | .900 | .0091 |
| | 2250. | 366. | .372 | .876 | .0155 | 346. | .347 | .884 | .0130 | 327. | .326 | .891 | .0111 |
| 7.1 | 2500. | 430. | .397 | .868 | .0184 | 407. | .372 | .876 | .0155 | 386. | .350 | .883 | .0132 |
| | 3000. | 564. | .441 | .853 | .0245 | 537. | .415 | .862 | .0208 | 511. | .392 | .869 | .0178 |

*"Balanced steel ratio" applies to problems involving bending only.